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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 12, NO. 1 ▽ MARCH, 1931

AN OLD-TIME TOLLHOUSE

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PUBLIC ROADS

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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VOL. 12, NO. 1 MARCH, 1931 G. P. St. CLAIR, Editor

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TOLL ROADS

A STUDY OF THE HISTORY AND PRESENT STATUS OF TOLL ROADS IN THE UNITED STATES AND OTHER COUNTRIES

Reported by H. H. KELLY, Division of Highway Transport, United States Bureau of Public Roads

THE conception of roads as free means of communication has been fundamental in man's economic thought. Imposition of direct tolls upon travelers as a means of paying for usage of the road, wherever it has occurred, has been of comparatively short duration, and to-day exists in only a few places in the world.

Highway tolls, levied for the purposes of constructing or maintaining roads, are not to be confused with customs duties or octroi taxes, which in all ages have been imposed upon goods transported for sale as a means of raising revenue or protecting domestic interests. Neither are they to be confused with the fees exacted from travelers as payment for safe passage of themselves and their goods, a practice universal in Europe in the Middle Ages and still to be found, for example, in the bandit-ridden areas of interior China.

SUMMARY OF ESSENTIAL FACTS AND CONCLUSIONS

Public sentiment and policy, which traditionally regard the highway as a free means of communication, are generally antagonistic to the toll-road principle.

The total length of toll roads now existing in the world is negligible—in the United States perhaps 150 miles, as contrasted to 3,000,000 miles of public roads.

Where public need for express highways is greatest, they are being built successfully without recourse to the toll method of financing, and the trend is toward increasing development of such free superroads built and operated at public expense.

Present systems of taxation place the major burden of highway construction and maintenance upon the highway users almost as directly as by the toll method, and in a much more equitable and acceptable manner.

Current proposals for toll roads in the United States generally appear to ignore the basic economic factors involved. The problem of toll roads is first and foremost, economic; and second, technical. The engineering difficulties are not insuperable; the economic difficulties, however, have not yet been satisfactorily solved.

Under present conditions, there would seem to be no justification for toll roads in any highly-developed country where a large mileage of public roads already exists; however, in undeveloped regions of the world some basis might be found for employing the toll method as the first step in financing a trunk-line system of highways.

EARLY HISTORY RECORDS FEW TOLL ROADS

There is little evidence in antiquity of the use of the toll system upon roads. Probably the earliest recorded instance of the collection of tolls was upon the great Persian military road leading from Babylon to Syria, as reported by Strabo, the date being approximately 2000 B. C. (*1*).[1] Under Babylonian law, every city of that great commonwealth had its own customs, ferry dues, and highway and water rates, levied on all but its own citizens (*2*). Levying of tolls on caravans passing through Palestine was part of King Solomon's fiscal operations, which were based on the systems prevailing in neighboring States, especially Egypt (*3*). It is significant, however, that in all these early instances the tolls were imposed either for military purposes or as a means of taxing foreign travelers and goods. On the other hand, the Appian Way, most famous road of classical times (begun in 312 B. C.) and other Roman highways, built primarily for military purposes, appear to have been free for public use when this did not interfere with the business of Empire (*4*).

The first actual record of a toll road or "turnpike," in the true meaning of the term,[2] was in England in the year 1346, when King Edward III granted permission for levying toll on all passing from St. Giles's to Temple Bar, London, for the reason that roads there had become impassable for want of other provisions for their maintenance (*5*).

Three hundred years later, in 1663, the first English turnpike act was passed, giving a franchise to the "Great North Road to York and Scotland" and permitting collection of tolls as a means of providing funds for maintenance. Coincident with the industrial revolution, the need for improved means of communication caused formation of numerous turnpike companies, state funds not being available for road purposes. At the time of the death of the great road builder McAdam in 1836 English turnpikes had reached their maximum development, with 1,100 turnpike trusts operating 20,000 miles of road; he himself had contributed largely to this result by his services in improving road-construction methods. Shortly thereafter, the decline of turnpikes began with the advent of railroads.

France at various times employed the toll system, the Government professing to devote the proceeds to the improvement of roads. The tolls gave rise, however, to gross peculations and widespread abuse, barriers springing up on all the roads and bridges and at the gates of towns, which one could not pass without making payment. Napoleon I finally abolished the tolls about the year 1806 (*6*) (*7*).

In the United States, the first toll road was built in Virginia in 1786 to carry the increasing traffic from Alexandria—which Washington had hoped to make one of the principal ports of the new republic—to northwestern Virginia and the navigable streams flowing toward the west. It was known as the Little River Turnpike. Similar roads appeared in Connecticut in 1792, Maryland in 1793, Tennessee in 1801 (through Cumberland Gap), North Carolina in 1804 (through the Cherokee lands). Necessity for expensive repairs on poor-surfaced roads which were subject to ever-increasing traffic and for tracing new routes westward, together with the lack of public funds, made inevitable the use of the toll system.

An outstanding example of American turnpikes was the Philadelphia and Lancaster Turnpike Road, the first macadam road in the western hemisphere, built 1792–94 at a cost of $465,000. It was 62 miles long, macadamized throughout with stones broken to pass through a 2-inch ring, and was guarded by nine toll-

[1] References to the bibliography at the end of the article are indicated by italic numerals inclosed in parentheses.

[2] Webster's dictionary supplies the following definition: "Turnpike road.—A road that has, or had formerly, turnpikes, or tollgates, established by law to collect from users tolls to defray the cost of building, repairing, etc."

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gates. It followed the location of the "King's Highway," a public road which had been constructed in 1741. Rates of toll ranged from 1 to 13½ cents per mile for each vehicle, depending upon the width of the tires and the number of horses used. This road enjoyed 25 years of prosperity, with annual dividends sometimes exceeding 15 per cent, and its success led to the formation of many similar enterprises. By 1811 there were 137 chartered roads in the eastern United States, with a total of 4,500 miles under operation, capitalized at $7,500,000 (8).

Toll Roads Served the Traffic of an Earlier Day

GREATEST DEVELOPMENT IN EARLY NINETEENTH CENTURY

In 1808, Albert Gallatin, Secretary of the Treasury of the United States, made a report on "Turnpikes, or Artificial Roads" in which it was stated that "great numbers" of such roads had been completed in eastern and southern States at costs varying from $1,000 to $14,000 per mile. Turnpikes were then in existence in Connecticut, Massachusetts, New York, "where in less than seven years 67 companies have been incorporated with nominal capital of near $5,000,000 for the purpose of making more than 3,000 miles of artificial road," New Jersey, Pennsylvania, Maryland, and Virginia.

The famous National Pike, the only highway of its kind ever wholly constructed by the Government of the United States, was a toll road, with commissioners named by the various States through which it passed to administer the toll collections. The main section of the road as laid out by the Federal Government was from Cumberland, Md. (from which the name of "Cumberland Road" often applied to it, was derived) to Wheeling, W. Va., via Uniontown, Pa., although the highway continued on westward to lose itself in the prairies of Illinois. The first definite proposal for construction of this road came in 1806; it was opened to the public in 1818; and was reconstructed in 1832. Tolls were collected at certain points on this road as late as the opening of the present century.

The peak of turnpike development occurred about 1830, and the decline began soon afterward with the rapid extension of railroads and canals (5).

MODERN TOLL ROADS

Compared with the total mileage of roads in the world to-day, the extent of toll roads is infinitesimal. Even in Europe, where unusual postwar conditions might have been thought to justify the use of tolls at least temporarily, there are few toll roads. Despite such special circumstances as the poor condition of public highways following the war, lack of public funds for large reconstruction programs, the considerable volume of de luxe tourist traffic, and the large numbers of animal-drawn vehicles and bicycles which hamper the movement of motor cars on the public roads, serious consideration has been given the toll-road principle in only a single country.

Italy.— Foremost among modern toll roads are the autostrade of Italy, principally located in the northern part of the country. The first autostrade, connecting the city of Milan with the lake resorts of Como, Varese, Arona, and Bergamo and representing a total length of 85 miles, were begun in 1925 by a private company

[4] The assistance of the Bureau of Foreign and Domestic Commerce of the United States Department of Commerce in supplying data regarding modern toll roads, particularly those in foreign countries, is gratefully acknowledged. The American Automobile Association also supplied valuable information on toll roads in the United States.

known as the Societa Anonima Autostrade, capitalized at 50,000,000 lire (about $2,500,000). The moving spirit behind the enterprise was the engineer Puricelli, head of one of Italy's most important construction firms. These roads are now in year-round operation. As an example of the toll charged, the round-trip rate between Milan and Como (46 miles) for a medium-sized automobile is about $1.25, with seasonal tickets available at reduced rates. The roads are of modern construction, principally 4-lane width, built of concrete with bituminous surface treatment, without grade crossings or intersections, and without speed limit save for trucks. Only rubber-tired passenger cars and trucks and motor cycles with side cars are permitted upon them. Other autostrade have since been built from Naples to Pompeii, from Florence to the coast, and in part from Milan to Turin.

The Milan-to-Lakes autostrade cost $80,000 per mile. The method of financing was by constitution of special companies for each main section of the highway and by issue of obligations guaranteed by the Italian Government, which in compensation for this service becomes owner of the highway after 50 years without making any disbursements (*9*).

During the first year of operation, revenues were barely sufficient to meet carrying charges; the second year a 2 per cent dividend was declared, and modest dividend disbursements have been made annually ever since. Recently, however, the improvement and extension of the national highways have diverted much traffic from the toll roads. For example, the national road from Milan to Como, having been resurfaced, is now more popular with motorists than the autostrada. This fact, together with the general business depression, which has reduced both local and tourist traffic, has given rise to general doubts that these toll roads will ever prove the lucrative venture which they were originally hoped to be.

Germany.—The only toll road in Germany is the Avusbahn, 12½ miles long in a closed circuit, between Berlin and Charlottenburg. It was built in 1921, chiefly for test or racing purposes. The round-trip toll is 50 cents. For several years, a proposal to build a series of toll roads—Hamburg-Basel, Stettin-Berlin-Munich, Dusseldorf-Cologne—has been given occasional publicity, but no definite action has yet been taken (10).

England.—During discussion of the road bill in Parliament in 1929, the fact was brought out that there were 55 toll roads in England and Wales at that time, all of a local character, and for the most part under 1 mile in length. Passage of this bill, which is now known as the road traffic act of 1930, permits of the compulsory transfer of such roads from their private owners to the public authorities, if necessary (*11*) (*12*).

Switzerland.—Tolls were charged in former years on a number of mountain roads where construction costs were very high, but these were abolished by law throughout the Swiss Confederation in 1928.

Austria.—Construction work was begun in 1930 upon a 30-mile road across the Grossglockner range, between Ferleiten and Heiligenblut, to cost about $1,750,000 and to be completed in three years. It will be one of the highest roads in Europe, with a maximum elevation of 8,200 feet. Present plans are to operate it as a toll road, charging about $3.50 per automobile.

In other countries of Europe, there doubtless are roads of local interest upon which tolls are charged, but accurate information on this point is lacking. In no case, however, are tolls charged upon a traffic artery of national importance. (Indicated by reports to International Road Congress.) In one or two countries there is agitation for express highways; in France, for example, a Compagnie des Auto-Routes has been formed to push a project for an express route from Paris to Lille and Calais, cost of which is estimated at 1,400,000 francs per kilometer (close to $90,000 per mile). Public sentiment, however, appears to be unenthusiastic about the creation of toll roads and to prefer the Government itself to build any superhighways of this kind.

The Fork at Lainate, on the Milan-Lakes Autostrade

Peru.—A toll road is under operation between Huaral and Huacho, primarily as means of access to a pleasure resort; and another 25 miles long, between Lima and Ancón, is partially completed. Concessions for highway construction granted under the recently deposed Leguia régime have largely been canceled by the new Government.

Brazil.—A 25-year concession was granted in 1929 for construction of an automobile toll road from São Paulo to Santos, to cost $5,000,000. At the end of 25 years, the road is to revert to the State without cost, and at any earlier date may be taken over by the State upon payment of cost plus 15 per cent (*13*). There are reported to be a number of local roads operated elsewhere in Brazil under the toll system.

Argentina.—The Government at the end of 1930 called for bids upon a project which, if it materializes, will be the largest toll-road enterprise ever undertaken. Two highways are proposed, one northwest from Buenos Aires to Rosario and Cordoba, and the other southwest from Buenos Aires to Azul and Bahia Blanca. The combined length of the two projects is about 800 miles. These toll roads must parallel free roads, but the latter are of low type and not passable at all seasons of the year. The period during which tolls may be imposed must not exceed 15 years, at the end of which time the highways will become the exclusive property of the Argentine nation. As protection for the constructing and operating company, if net receipts are not sufficient to cover interest and amortization on the capital investment, the Argentine Government will grant the company a subsidy sufficient to make up the difference; if net receipts are greater than the interest and amortization charges, however, the surplus must be given to the Government. The chief uncertainties in the project appear to lie in the legal technicality that the present Argentine Government is "provisional," and its power to enter into an agreement of this kind and support it

with adequate guarantees is questioned, and in the fact that the national constitution seems to insure the freedom of all highways.

FEW TOLL ROADS REMAINING IN UNITED STATES

Hardly more than a score of toll roads are known to exist in the United States to-day, all of them being privately owned. Only one of them can be said to attract traffic by the special facilities it offers, while the others are of a scenic character, of interest almost exclusively to tourists and sightseers. The total length of all these United States toll roads is estimated at less than 150 miles, as contrasted to the 3,000,000 miles of the public road system. An official inquiry to the highway departments of all the States of the Union early in 1931 revealed that there are no toll roads of any character in 35 States. The following paragraphs describe briefly the few toll roads now in operation in the other States.

Arizona.—The San Francisco Mountain Boulevard is a toll road extending from the vicinity of Flagstaff to the summit of San Francisco Mountain nearby. The toll charge is $1 per automobile.

California.—Six toll roads, on only one of which there may be said to be arterial traffic, exist in California. They are as follows: Darwin, easterly to Death Valley, toll charge $2; Mount Diablo, toll charge $1.25; Tamalpais Mountain, toll charge $1; Sears Point Road, which has some value as a traffic artery although there is an alternate free road through Napa which is only a few miles longer; Mount Wilson, and Seventeen-Mile Drive.

Colorado.—The Pikes Peak Auto Highway leads to the 14,000-foot summit of Pikes Peak, Colo. It was constructed in 1915 by E. A. Sunderlin, of Colorado Springs. The company owning this highway has offered to convey it to the United States Government at the end of 1935, and the offer has been accepted (*14*) (*15*).

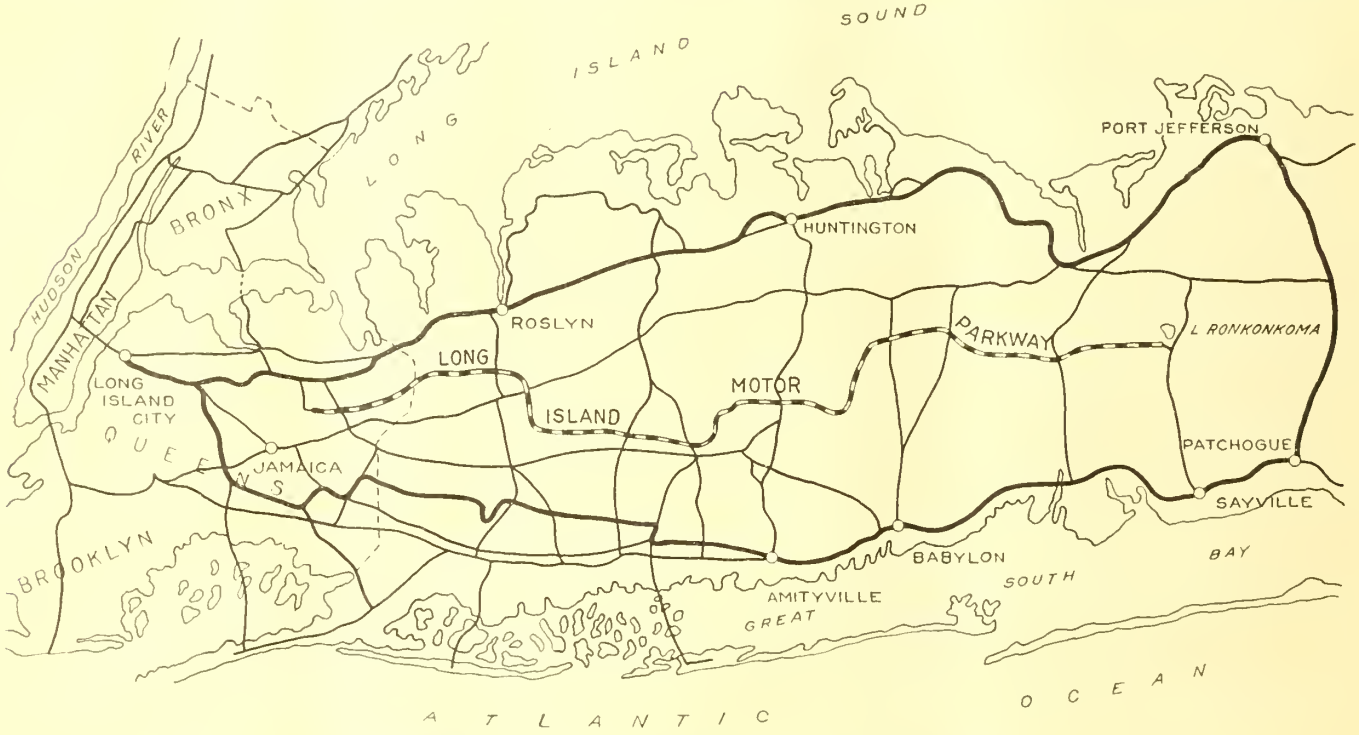
Florida.—Tolls are collected on the Beach Road between St. Augustine and Daytona, a thoroughfare used principally for its scenic value.

New Hampshire.—The Mount Washington Summit Road, 8 miles long, extends from the Glen House at an elevation of 1,543 feet to the summit of Mount Washington, elevation 6,293 feet. The franchise to this company was granted by the legislature in 1859 and the road is still operated under this original franchise.

New Jersey.—Tolls are collected on a road between Ocean City and Longport, this road forming an approach to a toll bridge, and connecting with two public roads.

New York.—The best-known toll road in the United States is the Long Island Motor Parkway, a privately owned highway handling automobile traffic of a restricted character; that is, pleasure cars to the exclusion of trucks and other commercial traffic. It is about 45 miles long and extends from the congested metropolitan district adjacent to Flushing, Long Island, to Lake Ronkonkoma in the center of the Island. Construction of the road was commenced in 1908 and most of it was completed by 1910, although short stretches have been added from time to time since that year. The right of way for it, from 50 to 100 feet in width, was acquired by purchase or lease, the company that built it having no power of condemnation. As now constructed the actual roadway has a width of from 16 to 22 feet, surfaced with concrete or bituminous macadam; it has many curves, and although for a considerable distance from its westerly end intersecting highways are crossed by bridge or underpass, toward its easterly end there are several highway crossings at grade.

The company which built this road, and which still operates it, was organized by a group of automobile enthusiasts in the early days of motoring. Capital stock was issued to subscribers who paid more than $800,000 for it, and while the stock was never listed on



Map Showing Location of Long Island Motor Parkway

any exchange it had a fairly wide distribution; later, to complete construction, there was an issue of mortgage bonds to the amount of $1,000,000, and all of these bonds, together with more than three-fourths of the outstanding stock, are now owned by one man who was active in the original enterprise. The company has never paid a dividend and its bonds have long been in default as to interest payments, although for several years its receipts have exceeded actual operating expenses. Including with original cost the interest on borrowed money accrued and unpaid, this parkway to-day represents an investment estimated at between

MAIN TOLLGATE ON THE LONG ISLAND MOTOR PARKWAY, LONGEST TOLL ROAD IN THE UNITED STATES

$6,000,000 and $7,000,000. The rate of toll is $1 for a single 1-way trip. Statistics on the traffic passing over the road have never been made public, for reasons of company policy, but the volume is said to vary between wide extremes in summer and winter.

This parkway has not been a financial success. Its operation is continued chiefly because of the personal interest of the owner of the bonds and stock above mentioned and his hope eventually to justify in some measure the investment made in it.

The "Conditions" printed on the back of each ticket of the Long Island Motor Parkway are of documentary interest.

CONDITIONS

Every person exercising the license granted by this ticket consents and agrees in consideration of the granting thereof, to the following conditions:

FIRST. Each person entering upon the premises of the Long Island Motor Parkway (Inc.) by virtue of this ticket thereby assumes all risks of injury, damage, and loss to himself or his property while on said premises and will in no event hold the Long Island Motor Parkway (Inc.) responsible for any injury to his person or for any loss or damage to his property while on said premises, whether caused by the negligence of said Long Island Motor Parkway (Inc.), or by the negligence or willful acts of its agents or employees, whether such willful acts be done within the scope of their employment or otherwise.

SECOND. The license granted by this ticket will be so exercised as not to interfere in any way with the exercise of a similar license granted to others.

THIRD. Each person entering upon the premises of the Long Island Motor Parkway (Inc.), by virtue of this ticket will obey and conform to all State, county, and local laws and regulations relating to automobiles or their operation and all rules and regulations for the use and operation of the motor parkway and will at all times while on said premises exercise all possible care and prudence, to avoid exposure of himself and others to risk of damage, injury, or loss and will keep to the right of each white line along the center of the parkway and generally keep to the right of said center whether or not so marked.

FOURTH. Automobiles when stopping must clear pavement with all wheels. Automobiles shall not be stopped or parked on top of or under bridges or on approaches to bridges or on curves.

FIFTH. Picnic parties shall keep the parkway and adjoining land as clean as they find it by disposing of their waste paper and refuse before leaving. No fire shall be made in the wooded sections. No parking west of Huntington Lodge, except in parking enclosures designated by signs.

SIXTH. The following speed limits shall be observed: Forty miles per hour west of large warning sign at Wyandanch; 30 miles per hour east of said sign. Except, however, that speed shall be reduced as required for safety, on curves and wherever necessitated by traffic conditions, and except that the speed limit shall be 10 miles per hour at each grade crossing of a public road and the traffic on such public road shall be given the right of way.

SEVENTH. The police maintained by the Parkway Co. shall have entire charge of the traffic on the motor parkway and the movement of cars thereon shall be subject to their direction.

EIGHTH. The company reserves the right to take up this ticket at any time.

The Bear Mountain Bridge Road, privately owned, which is the eastern approach to the bridge from Peekskill, N. Y., might be termed a "toll road," since the toll-gate for the bridge is located upon it

North Carolina.—Mount Mitchell Toll Road leads to the summit of Mount Mitchell, near Asheville, N. C. It was established in 1920 on the location of an old lumber railway. It is 18½ miles long, and wide enough for only one-way traffic. The toll rate is $1 for adults and 50 cents for children.

South Carolina.—A toll road about 12 miles long leads from Charleston to Folly Beach, a seaside resort of Charleston County.

Texas.—Two toll roads are in operation in the region of Medina Lake in southern Texas, carrying recreational traffic from the public roads to the lake. The first was opened in 1917 by the Medina Lake Toll Road Co.; its length is 7 miles, its surface is oiled gravel, and it cost $66,000. The toll rate is 50 cents each way for passenger cars, and $1 per ton each way for trucks. The second road was opened in 1927 by the Medina Highland Toll Road Co.; its length is 9 miles, its surface is gravel and graded earth, and it cost $75,000. The toll rate is the same as on the first road. Volume of traffic on both roads averages from 20 to 30 cars per day.

Utah.—Fees are charged for the issuance of automobile and motor-cycle permits on a road traversing the southeast corner of Zion National Park, the charge per automobile being $1. Residents of the counties adjoining the park are exempt from payment of this fee when in pursuit of their usual business.

Vermont.—The Mount Mansfield Toll Road, approximately 3¼ miles long, permits tourists to visit the summit of the mountain.

Virginia.—One of the few relics of the numerous toll roads of a century ago is the Great Falls Toll Road, leading to Great Falls, Va., near Washington, D. C. Its traffic is now limited principally to tourists, picnickers, and others wishing to visit the falls of the Potomac River, where there is a small pleasure resort. The toll rate is 25 cents per car.

An example of a toll road recently taken over by public authorities is the Conners Highway, a 51-mile road from West Palm Beach to the northern shore of Lake Okeechobee, Fla. Originally built and financed by W. J. Conners, of Buffalo, its outstanding obligations were assumed by the County of Palm Beach and it was freed from tolls in 1930. It is to be incorporated in the State highway system.

During the last five years, the public press has reported numerous suggestions for the construction of toll superhighways, but none of these has ever been actually undertaken. Among the specific locations mentioned have been the following: From One hundred and thirty-eighth Street in New York City to the village of Peekskill (elevated structure); from the New Jersey side of the Holland tunnel along the Atlantic coast to Atlantic City (semielevated); Atlantic City to Philadelphia; Detroit to Pontiac (elevated); San Francisco to Los Angeles; the Merced River Valley in California; Wilson River in Oregon; Buffalo to Niagara Falls; Milwaukee, Wis., to Gary, Ind.; New York to Boston. For most of these, no definite projects have been advanced, and public sentiment as reflected by the press has shown itself unfriendly to the toll feature of these suggested highways.

Probably the most ambitious of all toll-road plans advanced in this country is that of the Union Highways Association for a nation-wide express motorways system. This association was incorporated in 1925, with headquarters in New York City, to encourage formation of "an express motorways system which eventually will spread over the United States, Canada, and Mexico, and link up all of the nations of this hemisphere." It proposed to form 42 individual motorways corporations as members of the Union Highways Association, each of which would build and operate a separate section of highway under the toll plan until its cost was entirely paid off, when it would revert to the State. The 42 highways involved would include one transcontinental route from east to west, two routes roughly paralleling the Atlantic and Pacific coasts, and a compact network of roads between the Middle-Atlantic and Middle-Western States.

To promote the first of these 42 suggested highways,[4] the New York and New England Motorways Corporation was incorporated under the laws of the State of New York. The scope of this first motorways project is indicated by the fact that the corporation estimates that the cost of the 65-mile section between The Bronx (New York City) and New Haven, Conn., will be $200,000,000.

These toll projects are not to be confused with the activities of various organizations whose purpose is to encourage the building of publicly-owned express highways. An example of such organizations is the National Highway Association, incorporated in 1912, the aim of which is the construction of 50,000 miles of national highways "to be built and forever maintained by the United States Government." With somewhat the same ultimate object in view, a joint resolution known as the Phipps-Robsion resolution was introduced in Congress in 1929, "creating a commission to study proposals for a national system of express motorways." As attested by documents supporting the resolution and by the statements of the authors of the resolution, only publicly owned highways were under consideration, and the use of tolls as a means of financing was not mentioned. This resolution was passed promptly by the Senate, but is still awaiting action by the House of Representatives. In 1929, a bill was introduced in the Senate (S. 309) providing for the construction of a post road and military highway from the Atlantic to the Pacific coasts, to be known as the Central Highway, in which the interesting provision was made that portions of the right-of-way could be leased to provide additional funds.

DEDUCTIONS FROM THE FACTS

From the evidence of past experience, certain conclusions may be deduced:

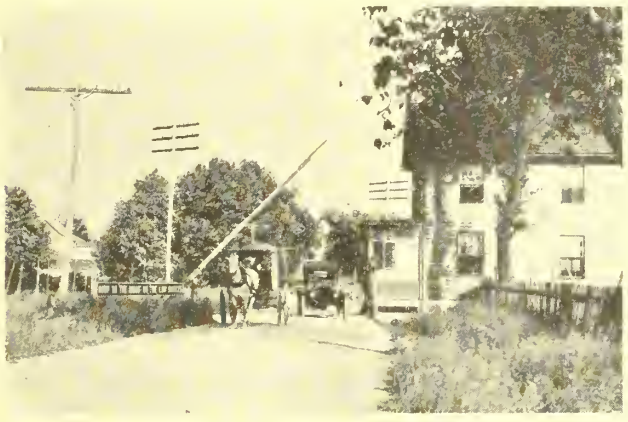
1. Reasons for the establishment of toll roads in the past have been: Nonexistence or inadequacy of free public roads, lack of State funds for construction of free roads to meet the public demand, and necessity of raising emergency revenue for the State treasury. Only the first two of these reasons appear applicable to modern conditions; and they are usually cited as the basis for current agitation for toll motorways in the United States.

2. Conditions under which toll roads have been successful are: Economically strategic location, assured volume of traffic, reasonable rate of toll, and absence of competition by free roads or other cheaper means of communication. Lack of any one of these basic conditions has been sufficient to make toll roads unsuccessful. Thus, even at the peak of toll-road development in the United States and Great Britain during the early part of the nineteenth century, numerous toll roads failed because of their poor location and consequently

[4] The information given here and in subsequent pages regarding this proposed motorway was obtained from literature issued by the promoters.

insufficient traffic. Moreover, the most successful toll roads (e. g., the Philadelphia-Lancaster turnpike) went into rapid decline as soon as competition by railroad and canal appeared. And the Italian autostrade, largest and most recent of such enterprises, appear to have already entered upon a period of diminishing returns because the improvement of free public roads serving the same territory has impaired their monopolistic character.

3. The possible advantages of toll roads may be summarized as: The relief of existing congestion on roads by early accomplishment of large highway improvements which the State in its normal program might not be able to undertake until considerably later; placing the burden of cost of the road directly upon its users, thus insuring that both foreign and local traffic pay an



A New Jersey Tollgate in 1911

equitable share in the facilities which they enjoy; and potential savings in time and cost of vehicle operation, as well as reduction of accidents.

4. The disadvantages of toll roads are: Antagonistic public opinion, which traditionally regards the roads as free means of communication; additional expense of financing immediate huge expenditures; additional expense of collecting tolls (a "nonproductive" activity which in the case of toll bridges has been estimated to absorb from 15 to 27 per cent of the total amount of tolls collected); impossibility of giving monopolistic guarantees, since paralleling free roads must be maintained for use by the general public, and consequent uncertainty of traffic volume; finally, that inherent dilemma in enterprises of this kind which presents on the one hand the necessity of building at heavy expense a superhighway capable of attracting large volumes of traffic from the free thoroughfares, and on the other hand the prime requirement of a low toll rate.

The above considerations apply to toll roads constructed and operated by public authorities. They are also applicable in all respects to toll roads promoted by private capital, but with such grave additional disadvantages in the latter case as intensified unfriendliness on the part of public opinion, materially higher costs in financing and possibly in construction and operation as well, and risk of loss to investors.

THREE SPECIFIC CASES STUDIED

In the light of these basic considerations, three outstanding examples of the toll-road principle—two of them in operation, one contemplated—may be briefly analyzed.

1. The Italian autostrade were built by private enterprise, but with the guarantee and support of the Government, at a time when public roads had been practically ruined by the war. They, therefore, enjoyed virtual monopoly, as well as a strategic location and a good volume of traffic; the power of the Fascist Government insured them a friendly public opinion, and they became an object of national pride. The toll rate—approximately 2½ cents per mile—did not prove a deterrent to traffic in a country where there is heavy tourist travel and where ownership of automobiles is still largely confined to the wealthier classes. Despite these exceptionally favorable factors, the financial success of the autostrade has been much less than originally anticipated, and their future as business ventures facing the competition of improved free roads is problematical.

2. The Long Island motor speedway was built by private enterprise, largely on the initiative of a single individual whose prestige and personal interest are still regarded as chiefly responsible for its continued existence. Its construction almost a quarter of a century ago was favored by conditions which could not be duplicated to-day: Comparatively low property values along the right of way, lack of good public roads, and accessibility to the first large automobile market in the country, New York City. The toll rate—about 2¼ cents per mile—is no deterrent to its de luxe traffic.

Tollgate Still in Existence on the Great Falls Toll Road, Virginia

The character of this traffic, and that of the wealthy and highly-developed region it traverses, have given it a practical monopoly little affected by the improvement of free public roads nearby. Nevertheless, its operation has not proved profitable.

3. The New York and New England express motorway has not been built. Its first section would extend from The Bronx, New York City, to New Haven, Conn., a distance of 65 miles. Purchase of the right of way alone for these 65 miles, paralleling in large part the line of the New York, New Haven & Hartford Railroad, would cost approximately $69,000,000, and the entire cost of building this 65-mile highway is estimated at $200,000,000. This tremendous cost is explained by the proposed specifications of the roadway: Right of way 200 feet wide in rural districts, 120 feet in towns; four separate tracks for traffic, each 25 feet wide, surfaced with concrete or other high-type material; the two inner (no speed limit) tracks divided by a 5-foot gravel space and a fence; the outer tracks divided from the inner tracks by 20-foot gravel spaces; all crossings of railroads or public roads to be by underpass or overpass; all crossings through towns or cities to be on elevated structures, carried wherever possible on the tops of specially constructed buildings one to four stories high; all approaches and exits to be by

ramps. It is estimated that such a highway could carry 200,000 cars over the entire distance between the Bronx and New Haven in a period of 24 hours without congestion or delay.

As originally proposed, the speedway was to be built by a private corporation with recapture clauses permitting each State through which it passed to take possession of that portion of the motorway lying within its borders, whenever desirable. The motorways corporation was to be capitalized at probably $20,000,000, which would be raised by sale of stock to concessionaires of a bus line, servicing and fueling enterprises, and contractors and suppliers of materials. The remaining $180,000,000 would be raised by the sale of debenture bonds, participating in the net earnings of the motorways corporation equally with the stock. Ten per cent of the total cost, or $20,000,000, was to be set aside as financing expense. The promoters estimated that the net earnings of the motorway would be over $20,000,000 per annum, a return of between 8 and 12 per cent on the stocks and bonds, permitting the motorway to earn its entire cost within a 10-year period.

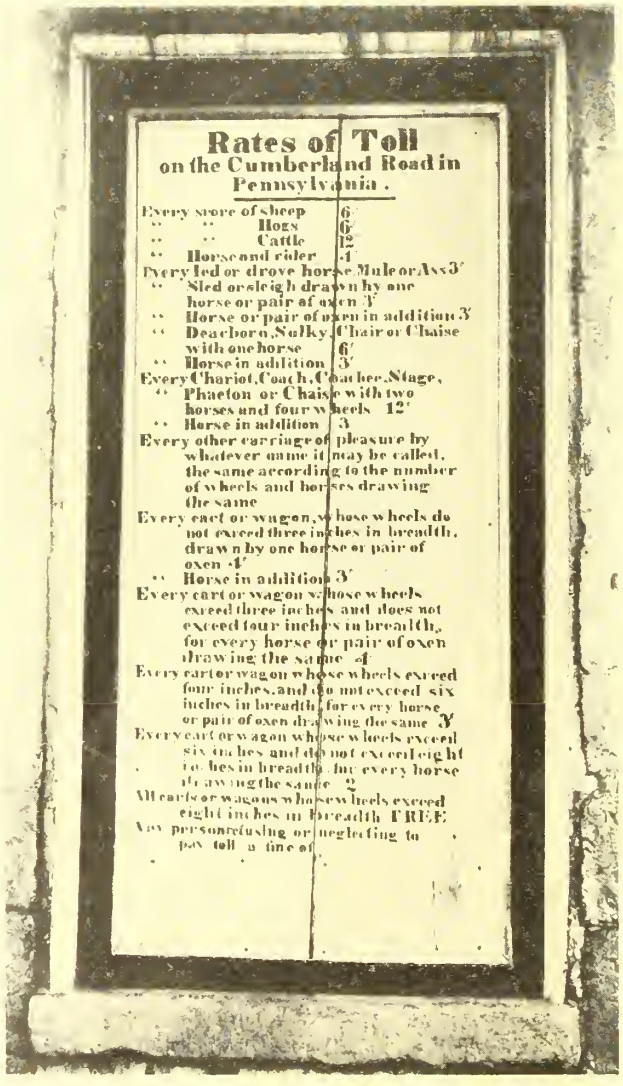
As these are estimated as the net earnings the gross receipts would necessarily have to be considerably larger. For a rough determination of the feasibility of the project, however, the same amount may be assumed to represent the gross receipts.

The following figures show the volume of traffic that would be required to produce a gross income of $20,000,000 per year which is equivalent to $308,000 per mile per year, or $844 per mile per day:

At 1 cent per mile toll	84,400 vehicles per day
At 2 cents per mile toll	42,200 vehicles per day
At 3 cents per mile toll	28,133 vehicles per day
At 4 cents per mile toll	21,100 vehicles per day
At 5 cents per mile toll	16,880 vehicles per day
At 6 cents per mile toll	14,067 vehicles per day
At 7 cents per mile toll	12,057 vehicles per day
At 8 cents per mile toll	10,550 vehicles per day
At 9 cents per mile toll	9,378 vehicles per day

An average of 84,400 vehicles per day over the entire distance of 65 miles, at 1 cent per mile toll, is beyond the realm of possibility for many years to come, and this minimum rate may therefore be regarded as entirely impractical. Probably the heaviest volumes of traffic in America to-day occur at the Holland Tunnel under the Hudson River at New York, and at the Delaware River Bridge at Philadelphia. The daily averages of vehicles passing these two structures during 1930 were respectively 32,930 and 33,660. The 2 cent and 3 cent rates, which roughly correspond to the present rates on the Italian autostrade and the Long Island Motor Parkway, may also be discarded as entailing too high a volume of traffic to meet expenses.

A partial measure of the actual current traffic between New York and New Haven is afforded by traffic counts made in 1930 at Milford, Conn., on the Boston Post Road, a modern 4-lane highway connecting the two cities, which show an average of 18,500 vehicles per day passing that point (annual average). (From T. R. Agg's report to Highway Research Council in December, 1930.) Assuming that all this traffic is through traffic between the two cities, and that the motorways would secure one-half of it, or 9,250 vehicles per day—both of which assumptions are entirely unwarranted, considering the large volume of local short-distance traffic and the excellence of the free Boston Post Road—the toll rate on the motorways would necessarily be at least 9 cents per mile, or $5.85 for the entire distance.


Rates of Toll
on the Cumberland Road in
Pennsylvania.
Every score of sheep 6'
" " Hogs 6'
" " Cattle 12'
" Horse and rider 4'
Every led or drove horse, Mule or Ass 3'
" Sled or sleigh drawn by one horse or pair of oxen 3'
" Horse or pair of oxen in addition 3'
" Dearborn, Sulky, Chair or Chaise with one horse 6'
" Horse in addition 3'
Every Chariot, Coach, Coachee, Stage,
" Phaeton or Chaise with two horses and four wheels 12'
" Horse in addition 3
Every other carriage of pleasure by whatever name it may be called, the same according to the number of wheels and horses drawing the same
Every cart or wagon, whose wheels do not exceed three inches in breadth, drawn by one horse or pair of oxen 1'
" Horse in addition 3'
Every cart or wagon whose wheels exceed three inches and does not exceed four inches in breadth, for every horse or pair of oxen drawing the same 2'
Every cart or wagon whose wheels exceed four inches, and do not exceed six inches in breadth, for every horse or pair of oxen drawing the same 3'
Every cart or wagon whose wheels exceed six inches and do not exceed eight inches in breadth, for every horse drawing the same 2
All carts or wagons whose wheels exceed eight inches in breadth FREE
Any person refusing or neglecting to pay toll a fine of

RATES POSTED ON OLD TOLLHOUSE ON THE NATIONAL HIGHWAY

In this connection, it is noteworthy that where public demand for express highways has been greatest, such highways have actually been built or are being built by public funds without recourse to the toll method of financing. Some of these highways would meet the most difficult requirements of the proponents of toll-supported superhighways. To cite only a single example, the Hutchinson River Parkway in Westchester County, N. Y., north of New York City, is a thoroughfare 14 miles long, without a single grade crossing, landscaped throughout in an elaborate and imposing manner. The Westchester County Park Commission which built this road also has under way a Pelham-Port Chester Parkway, for which the right of way has been purchased by county funds at a cost of $4,410,000. This public superhighway will have two separate roadways, each 44 feet wide (4-lane) and although it passes through a built-up area, it will be entirely free of grade crossings. Such projects clearly indicate the future progress of public highway development in regions where traffic is heaviest (*16*).

CONDITIONS FAVORABLE TO TOLL ROADS FOUND IN SOME REGIONS

The question naturally arises, "Are there any localities where present-day conditions might favor the creation of toll roads?"

In view of the essentials previously outlined for theoretical success of a toll-road project—economically strategic location, assured volume of traffic, reasonable rate of toll, and absence of competition—it is at once apparent that opportunities for successful toll roads in any such highly developed areas as the United States and western Europe are lacking, since nowhere could these basic desiderata be guaranteed.

In less developed regions of the world, where the need for improved roads is a vital one, and yet where public funds are not sufficient to finance extensive trunk-highway development, there are perhaps localities where the toll road would be both feasible and profitable.

The Argentine project, previously mentioned, to connect Buenos Aires with important cities several hundred miles distant, is a case in point. Although Argentina has the highest total automobile registration of any country in Latin America, and the highest per capita registration of any country in the Western Hemisphere outside of the United States and Canada, it possesses to-day less than 500 miles of hard-surfaced roads. There are now 365,000 automotive vehicles in Argentina, of which 35 per cent are concentrated in the municipality of Buenos Aires, with other large proportions in the districts of Cordoba and Bahia Blanca which the proposed toll road would serve. Here, then, is an assured volume of traffic and a strategic location; practical absence of competition is indicated by the low state of the free roads, which are passable only part of the year. With a conservative type of construction, and a reasonable rate of toll, the success of such a project is possible.

Similar possibilities might be found in other countries where the demand for improved roads already exists, and where the need for a start to be made on a national highway system is urgent. In such localities the toll road might prove worthy of serious consideration as the initial step in a highway program.

PUBLIC OPINION GENERALLY HOSTILE TO TOLL ROADS

In the final analysis, the toll road must answer for itself at the bar of public opinion. As one commentator put it, in reviewing the early development of turnpikes, "The capital supplying the turnpikes . . . was in fact, subjected to two risks: First, the ordinary business risk; and secondly, a risk incidental to the fear of arbitrary interference by a public whose preconceptions were of an unfriendly nature." (Frederic J. Whiting, in preface to Wood's "Turnpikes of New England.") That double risk still exists.

Motor vehicle manufacturers have a predominant interest in highway development. Their complete opposition to the toll-road principle was expressed in an official resolution adopted by the National Automobile Chamber of Commerce, which numbers among its members practically all of the passenger car and truck manufacturers of America, under date of February 1, 1928:

Whereas, the free use of the public highway is a fundamental principle of Government; and

Whereas motor vehicle owners of the United States through special taxes are to-day largely meeting the costs of the construction of these highways; and

Whereas so-called express highways are simply a step in the further improvement of our public highway system which will be found necessary in those localities where traffic is heavy and congestion great: Therefore, be it

Resolved, That the National Automobile Chamber of Commerce opposes any effort to place control of any part of the public highways in the hands of private promoters; and be it further

Resolved, That the National Automobile Chamber of Commerce emphatically supports the principle that the public highways shall be kept forever free to the general public (*17*).

The public will, as expressed in laws of Congress, has shown itself to be opposed to the toll-road principle. Under the Federal-aid road act of 1916, Federal aid for toll roads is prohibited, and all highways constructed or reconstructed under provisions of that act "shall be free from tolls of all kinds." In 1927, it is true, Congress passed an act permitting use of Federal aid funds on construction of toll bridges and approaches "providing that such bridge is owned and operated by the State or a political subdivision thereof, and tolls are used to repay the State and permit the freeing of the bridge." A sharp distinction was thus drawn between publicly owned and privately owned structures. To-day the only roads in the Federal-aid system still excluded by law from the benefits of Federal aid are those approaching a ferry or bridge built by private individuals or private companies (*17*).

A Tollhouse in Vermont, Photographed in 1912

From the international point of view, a significant change in the public attitude toward toll roads is evidenced in the proceedings of the Permanent International Association of Road Congresses. This association, headquarters of which are in Paris, is officially supported by more than 60 nations, and includes among its members leading engineers, traffic specialists, and government authorities from all parts of the world. At its fifth congress, held in 1926 in Milan, Italy, where the first autostrada had just been completed, the association passed resolutions encouraging the construction in all countries of highways reserved exclusively for motor traffic, and advocating the use of the toll system as a means of financing. The delegates of the United States and Great Britain abstained from voting on these resolutions, making a formal statement to the congress in which they described public sentiment in their countries as being opposed to the toll principle, and suggested that as regards the operation of toll-supported motor roads, "time has been too short to provide experience on which definite conclusions capable of general application can be based" (*18*).

Despite the action of the Milan Congress, the question of special motor roads was not deemed important enough to be included in the agenda of the next congress of the same association, held in Washington, D. C.,

in 1930. There was no discussion of the matter in the congress, and the various resolutions adopted by the delegates made no recommendation on the subject. However, in a preliminary report submitted by the delegates from Italy, who had been the chief proponents of the resolution in the Milan meeting, the following noteworthy statement was made, reflecting a much more cautious attitude toward toll roads than was the case four years earlier.

> . . . the autostrada can not expect to take all motor-vehicle traffic from the ordinary existing roads, even for journeys of equal length. . . . It is true that where the autostrade are of the same length or connect centers already served by ordinary routes which are to be rebuilt or improved, it is to be expected that, after the completion of the improvements, automobile traffic will prefer the good road which is free to the autostrada, which may be slightly better but on which a toll is charged. Since, on the other hand, the financial scheme of the autostrada is based principally upon the number of vehicles that will make use of it, there is danger of failure unless the ordinary road is or becomes so congested as to cause those motorists who want to make higher speed or enjoy a higher class of service to abandon it in favor of the autostrada. These considerations appear to us so striking that they can not fail to be recognized by the authorities and boards that consider the requests for concessions and by the statesmen who have the decision. We believe, therefore, that if a few concessions are granted under the above-mentioned circumstances, this presupposes an anticipated traffic density sufficient for the two roads, the ordinary and the special road, or an encouragement of the construction of the autostrada for the higher reasons of general benefit, independently of its economic success as an industrial enterprise (*19*).

The latest official pronouncement on the subject of toll roads, expressing complete opposition to the toll principle, is contained in the Official Report of the Royal Commission on Transport of Great Britain, issued at the end of 1930. A section of the report headed "Motorways" reads as follows:

> Periodically proposals are put forward for the construction by private enterprise of special roads reserved for motor traffic. The suggestion is that the necessary capital should be sought in the open market and that the promoters should look for their revenue to the tolls to be charged to users. The commissioners are strongly opposed to any suggestion of this nature. They have recommended the abolition of tolls and are not prepared to support any proposal which would have the effect of creating new tolls or prolonging the life of existing tolls (*20*).

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THE EFFECT OF VIBRATION ON THE PRESSURE OF CONCRETE AGAINST FORM WORK

Reported by L. W. TELLER, Senior Engineer of Tests, United States Bureau of Public Roads

FROM time to time during the past 35 or 40 years several investigators have made experimental determinations of the pressure of freshly placed concrete against the form work. As a result of experiments performed in 1894, Ernest McCullough found that the pressure obtained was equivalent to that of a fluid weighing 80 pounds per cubic foot.

Later experiments by other engineers indicated that the maximum pressure obtainable was equivalent to that of a fluid weighing 140 to 150 pounds per cubic foot. Among those whose efforts are noteworthy are F. R. Shunk,[1] E. B. Germain,[2] A. B. McDaniel and N. B. Garver,[3] and R. A. Sherwin.[4] Still later efforts were made to express in empirical formulas, terms which would take care of such variables as temperature and rate of fill. Paaswell[5] derived such a formula,

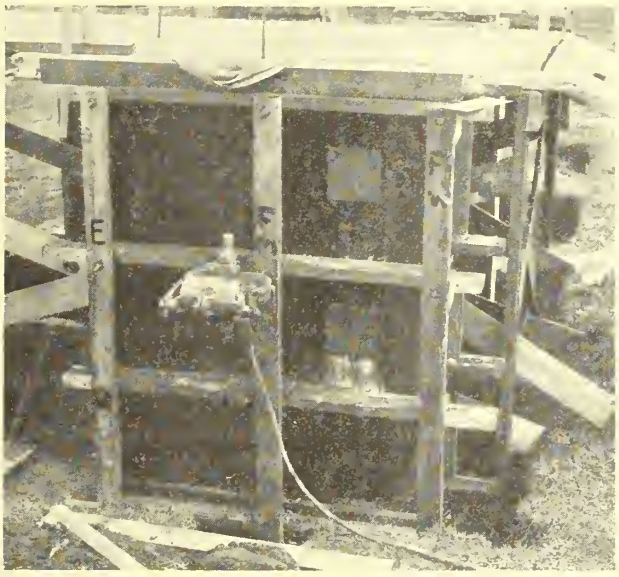

FIGURE 1.—ELECTRIC FORM VIBRATOR ATTACHED TO A FORM BATTEN

making use of the experimental work of Major Shunk a decade before, and E. B. Smith,[6] in reporting a series of tests by the United States Bureau of Public Roads, derived a formula based upon the data obtained in the tests reported by him.

While the conclusions reached by these investigators are not in complete accord, their work has been of great value to engineers in designing form work, particularly deep forms such as walls and columns.

Recently there has appeared a new factor which seems likely to have an important effect on the design of form work and that is the method of consolidating concrete by vibration. Several such methods are in use to-day and all of these tend to give temporary fluidity to very dense dry concrete mixtures which, as a rule, could not be placed without vibration.

In the course of a research project dealing with another subject at the Arlington Experiment Farm, Va., in the spring of 1930, an opportunity was presented to obtain a limited amount of data concerning the effect of vibration on the pressure of concrete against deep forms, and these limited data are being presented in this report in the hope that they may be of use to those who have this problem before them.

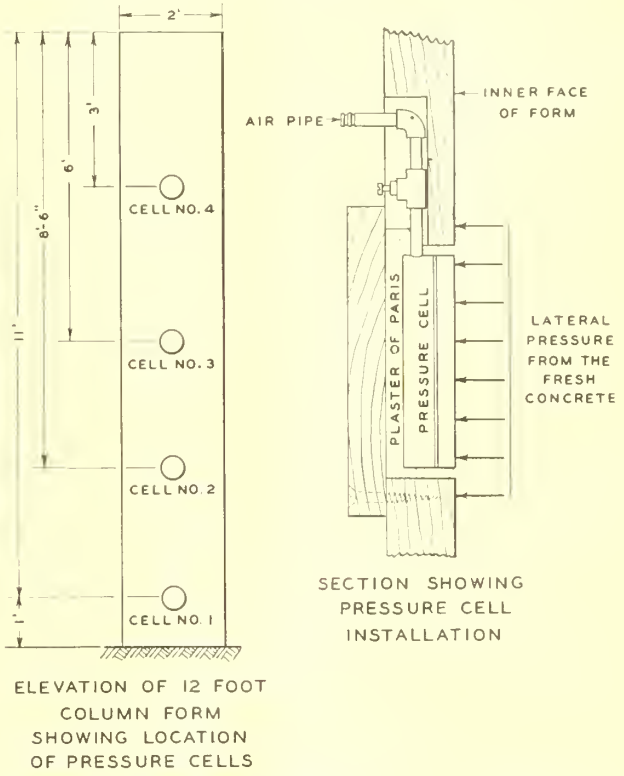

FIGURE 2.—DETAILS OF PRESSURE CELL INSTALLATION

TESTS MADE ON 12-FOOT COLUMNS

The forms which were used in these tests were for columns 12 feet high, 2 feet wide, and 8 inches thick. They were made of a 2-inch tongue and groove wood planking and were water-tight. The surface against which the concrete was placed was oiled.

The concrete used in all of the specimens was of the same proportions, aggregates, and grading, although the consistency varied with the different specimens. The proportions used were 1:2:4 by volume, with all materials batched by weight. The coarse aggregate was a well rounded local gravel with a maximum size of 1¼ inches and the fine aggregate was a fairly coarse sand from the Potomac River.

The rate of fill was nearly the same in all of the five columns on which pressure measurements were made and was approximately 20 feet per hour.

In two of the forms consolidation was accomplished by hand spading and, in these, concretes showing slumps of 3½ and 7½ inches, respectively, were used. The other three forms were vibrated and, in these,

[1] Pressure of Concrete on Forms, by Francis R. Shunk. Engineering News, vol. 62, No. 11, Sept. 9, 1909, p. 288.

[2] The work of Germain and McCullough is referred to in the paper by McDaniel and Garver. (See footnote 3, first reference).

[3] Pressure of Wet Concrete on the Sides of Column Forms, by A. B. McDaniel and N. B. Garver. Engineering News, vol. 75, No. 20, May 18, 1916, p. 932; and Wet Concrete Pressure on Column Forms, by the same authors. Concrete, vol. 10, No. 5, May 1917, p. 191.

[4] Designing Concrete Formwork Reduces Costs, by R. A. Sherwin. Engineering Record, vol. 73, No. 9, Feb. 26, 1916, p. 278.

[5] Pressure of Concrete in Forms, by George Paaswell. Engineering and Contracting, vol. 53, No. 8, Feb. 25, 1920, p. 209.

[6] Concrete Pressure Against Forms, by Earl B. Smith. PUBLIC ROADS, vol. 2, No. 23, March, 1920, p. 15.

concretes showing slumps of 1½, 3½, and 5 inches, respectively, were used.

The hand spading was very thoroughly done, being started as soon as the first concrete was placed in the form. The vibration was supplied by a special electric form vibrator clamped securely to the exterior of the form. This vibrator consists of an electric motor carrying an eccentric weight on the shaft and operating at the high frequency of 3,600 vibrations per minute. It is shown attached to a form batten in Figure 1.

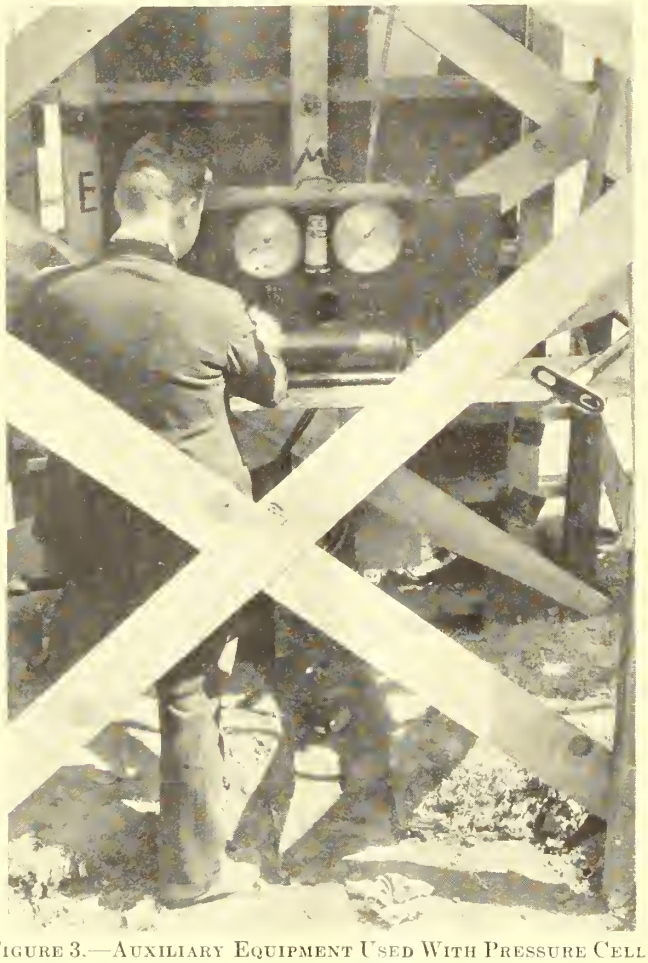

FIGURE 3.—AUXILIARY EQUIPMENT USED WITH PRESSURE CELLS

The tests were made during April and the temperatures were rather low (49° to 62° F.) except in the case of one section where the concrete was placed at a temperature of 74° F.

MEASUREMENTS MADE WITH SOIL PRESSURE CELLS

In order to measure the pressure of the concrete against the side of the column form four soil pressure cells, of the type developed by the bureau, were set into the side of the form at various heights as shown in Figure 2. This device consists of a flat cylindrical metal box, one face of which is slightly movable but is tightly sealed by a thin metal diaphragm. Normally the two faces of the box (ends of the cylinder) are in contact internally and this contact closes an electrical circuit. External pressure tends to maintain this contact. If air under pressure is introduced into the box or cell through the small pipe provided for the purpose, it will tend to expand the walls and when the internal pressure just exceeds the external pressure on the cell face, actual movement of one end will occur.

FIGURE 4.—GENERAL VIEW OF THE COLUMNS ON WHICH PRESSURE MEASUREMENTS WERE MADE

This movement interrupts the electrical indicating circuit and if the air pressure present in the cell at this moment is measured by suitable gauges, the external pressure on the face of the cell is determined. The movement necessary to break this circuit is extremely small and repeated tests have indicated that the device is thoroughly reliable, if properly installed and carefully used. The pressure cell and its use has been described in detail elsewhere.[7] The case containing the pressure gauges, air tank, control valves, and indicating light is shown in Figure 3.

Figure 4 shows several of the columns after the forms had been removed. The circular spots on the concrete are oil stains showing where the greased surfaces of the pressure cells were in contact with it.

The procedure followed in reading pressures was as follows: The gauges were first attached to the cell nearest the base of the form (cell No. 1; see fig. 2), and the pressure exerted against this cell was measured

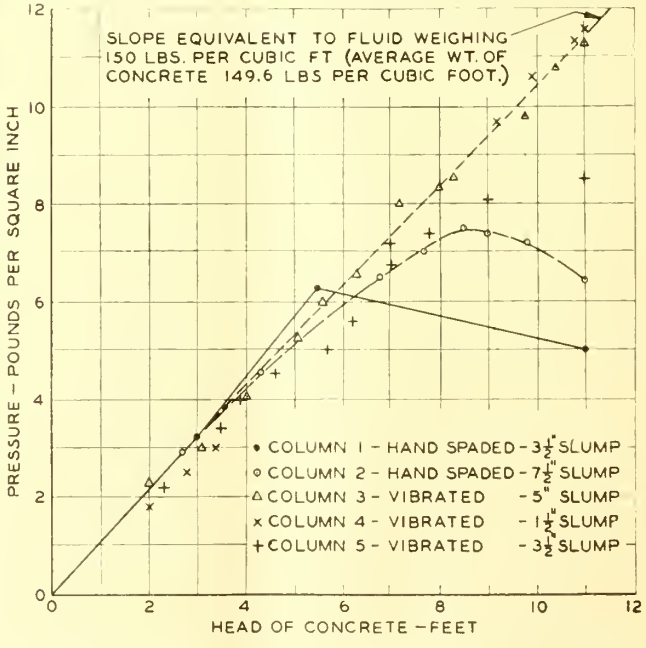
SLOPE EQUIVALENT TO FLUID WEIGHING
150 LBS. PER CUBIC FT (AVERAGE WT. OF
CONCRETE 149.6 LBS PER CUBIC FOOT.)
PRESSURE – POUNDS PER SQUARE INCH
HEAD OF CONCRETE – FEET
0
2
4
6
8
10
12
• COLUMN 1 – HAND SPADED – 3½" SLUMP
○ COLUMN 2 – HAND SPADED – 7½" SLUMP
△ COLUMN 3 – VIBRATED – 5" SLUMP
× COLUMN 4 – VIBRATED – 1½" SLUMP
+ COLUMN 5 – VIBRATED – 3½" SLUMP

FIGURE 5.—CURVES AND PLOTTED POINTS SHOWING PRESSURE AS A FUNCTION OF HEAD FOR ALL COLUMNS DURING PERIOD OF FILLING. DATA OBTAINED FROM CELL NO. 1 ONLY

[7] An Apparatus for Determining Soil Pressures, by A. T. Goldbeck and Earl B. Smith. Proc. Am. Soc. for Testing Materials, Vol. XVI, Part II, 1916, p. 309.

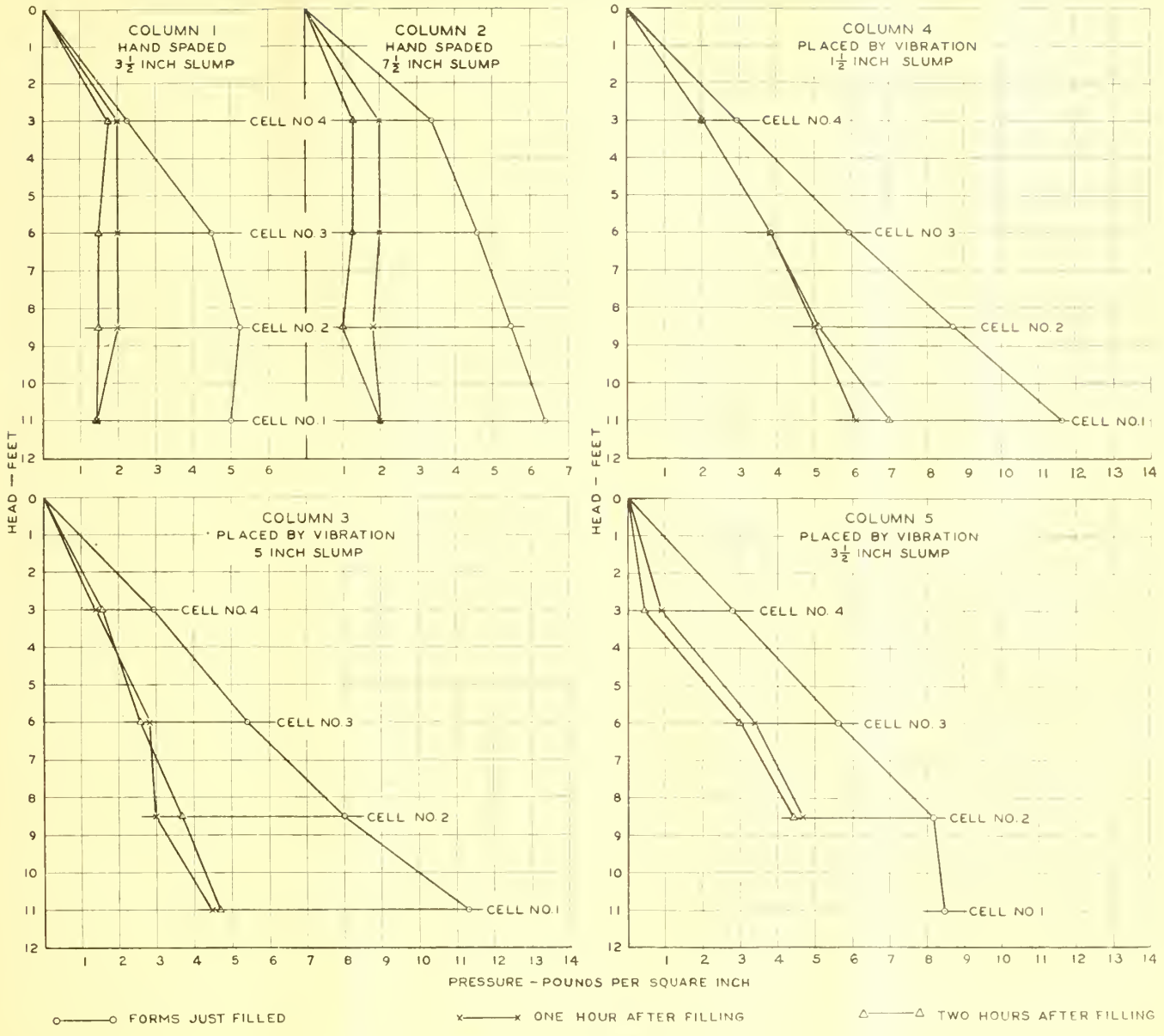

FIGURE 6.—PRESSURE DIAGRAMS FOR ALL COLUMNS

periodically as the form was being filled. The pressure, head of concrete, and the elapsed time since the filling started were recorded. As soon as the form was full, periodic readings on all four cells were begun. These observations were continued at 15, 30, or 60-minute intervals for several hours.

Practically all of the data obtained have been included in the graphs which accompany this report. It may be observed, however, that certain data pertaining to cell No. 1 and column 5 are not shown in Figure 6 and an explanation for this omission is offered. For some reason not entirely clear, there was noticeable segregation in the first batch of concrete placed in the base of this column. Perhaps this was due to the violence of the vibration, perhaps not. The result of the segregation was that the concrete in front of cell No. 1 in this particular column gradually lost its fine material and its fluidity and the pressure data for this cell were affected accordingly, as will be noted in the discussion later. The condition described can readily be seen in the photograph of the columns (fig. 4) in which column 5 is the one in the center, farthest from the camera. The fine material collected in the bottom of the form in a layer about 3 inches in thickness.

DATA SHOW FLUID CHARACTERISTICS OF VIBRATED CONCRETE

For the purpose or comparing the effect of the two methods of consolidation used, the pressure readings for cell No. 1 of each column during the period of the filling of the forms, expressed as a function of head of concrete, are grouped in Figure 5. It will be noted that, for the hand-spaded columns, even when concrete having a slump of 7½ inches was used, the concrete at the bottom of the form had ceased to act as a fluid long before the form was filled. The pressures increased hydrostatically up to a certain value, after which they decreased. This maximum pressure appears to increase with the fluidity of the concrete, as indicated by the slump test. In the case of the vibrated concrete, however, the fluidity was retained even for concrete having a slump of only 1½ inches so long as the vibration was continued.

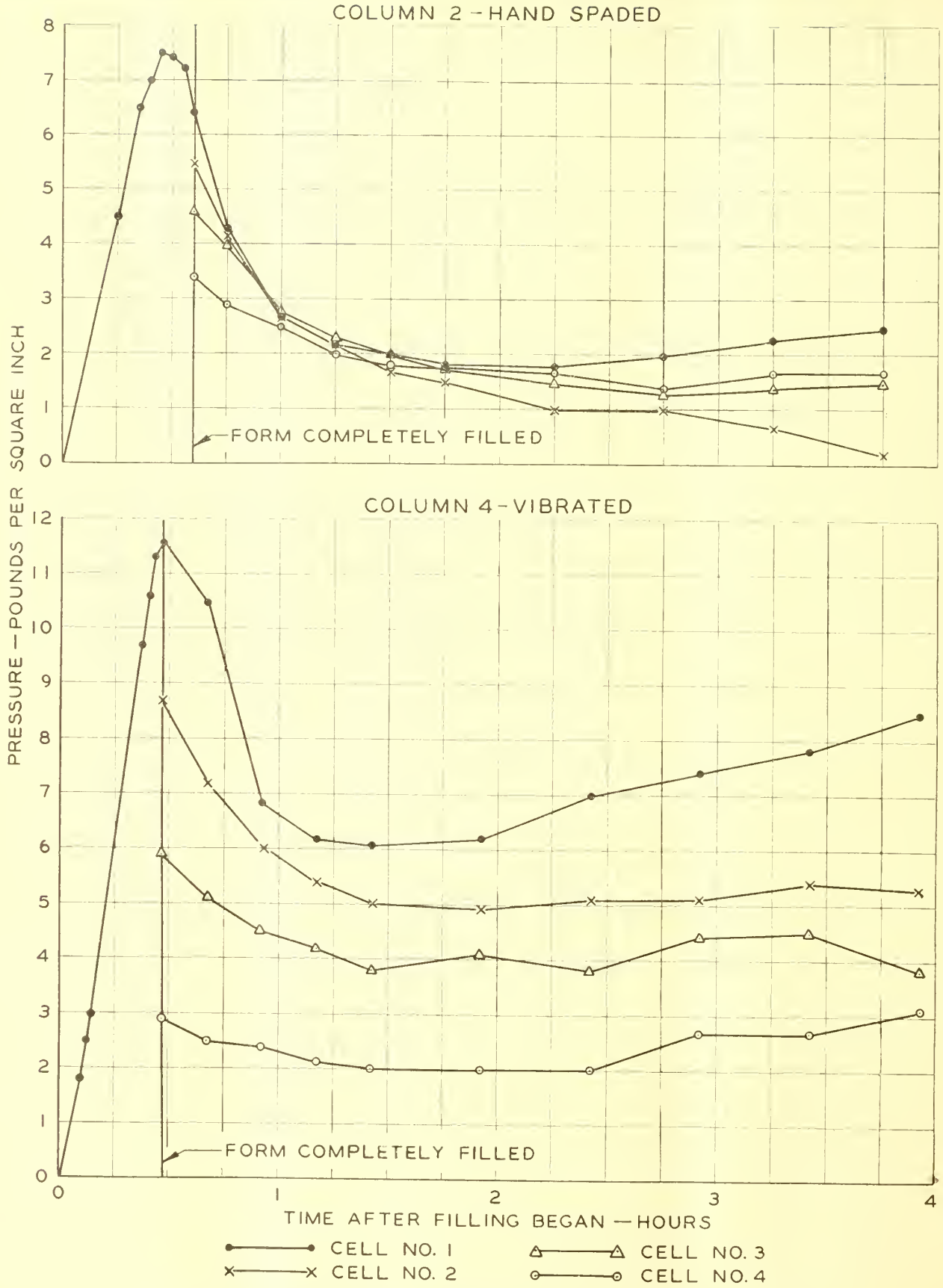

FIGURE 7.—CURVES OF PRESSURE AS A FUNCTION OF TIME FOR COLUMNS 2 AND 4

It is obvious that the complete fluidity of concrete with a 12-foot head is a matter for careful consideration in form design.

In Figure 6 are pressure diagrams for all five columns at three periods of time. Since the first curve for each column represents the pressure condition at the time the forms were just filled, the maximum pressure is not always shown. In the hand spaded columns the maximum pressures were recorded on cell No. 1 at some time before the forms were full. The rate and manner in which the pressure decreased during the first two hours after the concrete was placed is indicated by the diagrams. It will be seen that in the columns which were vibrated all of the pressure cells, except cell No. 1, column 5, indicate that the entire mass is acting as a fluid which weighs about 150 pounds per cubic foot at the time when the form is just filled.

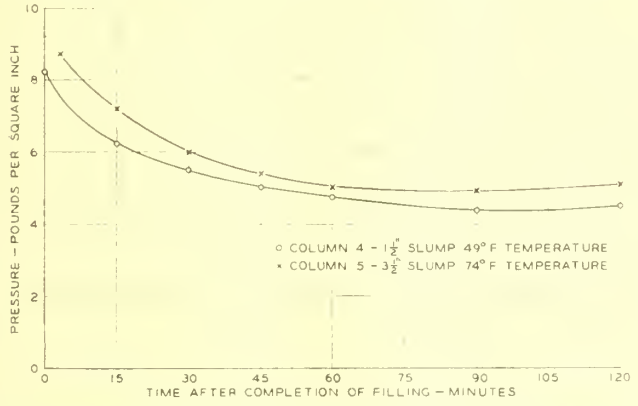

FIGURE 8.—CURVES OF PRESSURE AS A FUNCTION OF TIME FOR COLUMNS 4 AND 5, PLACED AT DIFFERENT TEMPERATURES. DATA OBTAINED FROM CELL NO. 2 ONLY

In both Figures 5 and 6, the effect of the segregation in the bottom of column 5 is noticeable. Since Figure 5 includes only the data from cell No. 1, the points shown for column 5 in this figure give an erroneous idea of the condition of the concrete in this column. The pressure diagram for column 5 in Figure 6 shows that the entire mass of the concrete above cell No. 1 was in a fluid condition at the time that the filling of the form had just been completed.

It is interesting to compare the pressure diagrams (fig. 6) for column 2 (hand spaded, 7½-inch slump) with those for column 4 (vibrated, 1½-inch slump) and note that vibration appears to be much more effective than water for obtaining a fluid condition in concrete.

While Figure 6 shows in a rough way the pressure variation with time for the different columns, in Figure 7 this relation for two of the columns (2 and 4) is given in more detail and for a longer period of time.

The most striking thing about these data is the great difference in the residual pressures resulting from the two types of consolidation.

After the 2-hour period there seems to be a tendency for the pressures to fluctuate, some increasing, others decreasing in a more or less orderly way. The cause for this is not known although it is thought that absorption of moisture by the wooden forms may have caused slight and gradual warping of the forms which, acting against the stiffening concrete, would produce such an effect.

TEMPERATURE EFFECT APPARENTLY NOT IMPORTANT

In order to determine whether there was any indication in the data of temperature effect on pressures, Figure 8 was prepared. These curves of time-pressure relation for columns 4 and 5 represent conditions in two specimens of concrete, both of which were in a fluid state at the completion of the filling of the forms. The comparison was made on these specimens because of the comparatively large temperature difference which obtained (25° F.) Since a comparison could not be made on cell No. 1 for the reasons previously described, the data from cell No. 2 on each column were used. Unfortunately for this comparison, another variable (consistency) is present, which may influence the relation to some extent. In spite of these factors, it is indicated that moderate variations in temperature did not greatly affect the pressure value or rate of change in these tests.

It is recognized that the tests are extremely limited in scope and no attempt has been made to draw any very definite conclusions. The data regarding the hand spaded columns are in general agreement with those of other investigators and those obtained on the vibrated columns are sufficiently conclusive to indicate the importance of giving consideration to this matter in the design of forms where consolidation by vibration is contemplated.

MOTION-PICTURE FILMS ON ROAD CONSTRUCTION AVAILABLE

A number of motion pictures have been prepared for the Bureau of Public Roads by the Office of Motion Pictures, United States Department of Agriculture. These pictures cover a variety of topics including the construction of various types of road, highway research, forest and national park roads, historical subjects, and road building in Latin America.

The purpose of these films is to acquaint the public with the methods and significance of highway work, to gain public cooperation, and to spread the knowledge of improved methods in the construction of roads. Persons or agencies desiring to borrow films should make application to the Office of Motion Pictures, Extension Service, United States Department of Agriculture, Washington, D. C., indicating specific dates and, if possible, alternative dates, on which they are desired for use. The films are furnished free of charge, except for transportation, which the borrower is required to pay both ways.

There follows a list of the available films, with a brief description of each.

TYPES OF ROADS

Low-Cost Road Surfaces—First Stage.—Indicates the savings in motor-vehicle operating costs made possible by low-cost road surfaces. Outlines the essential operations in the construction of the three representative types of low-cost untreated surfaces—gravel, sand-clay, and topsoil. (2 reels.)

Low-Cost Road Surfaces—Second Stage.—Shows that some form of surface treatment is usually necessary on low-cost surfaces where the traffic exceeds 500 vehicles per day. Describes the construction operations for the representative bituminous surface treatments as used in Florida, Tennessee, and South Carolina. (1 reel.)

Mixed-In-Place Bituminous Surfaces.—The methods employed in the construction of the mixed-in-place bituminous surfaces in the far Western States, the alternate "plant-mixed" type as used in California, and the retread surface as developed in Indiana and other Middle Western States. (2 reels.)

Penetration Bituminous Macadam.—Methods of equipment used in the construction of penetration bituminous macadam surfaces, which have reached a high degree of perfection in the New England States, particularly in Massachusetts and Rhode Island. (1 reel.)

Hot-Mixed Bituminous Pavements.—The essential features in the design and construction of "hot-mixed" bituminous surfaces, including coarse-aggregate asphaltic concrete, fine-aggregate asphaltic concrete, and sheet asphalt. (1 reel.)

Concrete Road Construction.—Illustrates the essential requirements in the design of concrete pavements to provide the necessary stability to resist the destructive action caused by the volume, weight, and speed of modern motor traffic. Shows the principal operations in the most advanced methods of concrete pavement construction. (2 reels.)

Brick—From Clay to Pavement.—The progress of clay through the plant until it is laid on the road as vitrified paving brick; some finished brick roads. Of general technical interest. (1 reel.)

Granite Block Paving.—From the quarry to the finished pavement in large cities. Of general technical interest. (1 reel.)

HIGHWAY RESEARCH

Impact of Traffic on Roads.—Tests made by the Bureau of Public Roads to determine accurately the effect on highways of the pounding tendencies of motor-vehicle wheels. Many tests in detail. Of special interest to road engineers. (1 reel.)

The Bates Road Tests.—The story in pictures of one of the most important road tests in highway history, depicting the methods by which a road was built to be destroyed. (1 reel.)

America's Great Bridge Test.—A technical study of the tests made to determine the strength of a modern reinforced arch bridge; made on the Swift Island Bridge over the Peedee River in North Carolina. (1 reel.)

FOREST ROAD BUILDING

Around the West by Forest Roads.—Examples of forest roads built by the Bureau of Public Roads in Colorado, Oregon, California, and Arizona. Of general interest. (1 reel.)

Building Forest Roads.—Men and machinery at work in the national forests, pushing good highways through the great mountains and woodlands. Of general interest. (1 reel.)

Highroads and Skyroads.—Building Government roads through the national forests; obstacles overcome and scenic beauties reached. Of general interest. (1 reel.)

Roads from Surf to Summit.—Scenic wonders of the national forests of the Pacific Coast States now revealed by modern highways: Lake Crescent in the Olympic National Forest, Mount Hood, Mount Rainier, Mount Baker, Mount Shuksan, and other natural wonders. (1 reel.)

Roads to Wonderland.—Scenic spots reached by roads that are being built by the Federal, State, and county Governments, Mount Hood in the Oregon National Forest; Crater Lake in Crater Lake National Park; and Yosemite National Park. Of general interest. (1 reel.).

The Road Goes Through.—How the western road builder overcomes barriers to transportation and builds the modern roads of our national forest and Federal-aid highway systems. (1 reel.)

NATIONAL PARK ROAD BUILDING

Roads in Our National Parks.—A panorama of the service performed by the Bureau of Public Roads in constructing modern highways in our Western National Parks. The scenic wonders of Yosemite, Glacier, Rocky Mountain, Mount Rainier, Mount Lassen, and Mesa Verde National Parks make this film one of the unusual beauty and interest. (1 reel.)

A Road Out of Rock.—How engineers and road builders overcome huge obstacles to construct a modern highway through Glacier National Park. Details of construction of a road which now brings the magnif-

icent "Garden Wall," picturesque Lake McDonald, and other scenic wonders of the park within easy reach of the tourist. (1 reel.)

The Men Who Build the Roads.—A tribute to the courage and ingenuity of the builders of our mountain highways; glimpses of the builders engaged in the task of penetrating the rocky barriers of Glacier National Park with a modern road. (1 reel.)

New Roads in Rainier.—A picturization of the work performed by the Bureau of Public Roads in improving the highways that make the natural wonders of Mount Rainier National Park so easily accessible to the public. Majestic Mount Rainier with its perpetual ice-capped peak and the 28 glaciers which clothe its sides, offer a scenic background of unusual beauty for this film. (1 reel.)

Rocky Mountain Park Roads.—The construction of the new road into Colorado's famous national park under the supervision of the Bureau of Public Roads engineers. Long's Peak, 14,225 feet above sea level, the Continental Divide, and picturesque Fall River Canyon, reached by this new highway, provided scenic material for the cameraman. (1 reel.)

Yosemite's New Roads.—The planning and building of new roads into the Yosemite National Park by the engineers of the Bureau of Public Roads. Yosemite Falls, 2,350 feet, highest in the world, El Capitan, and the exquisite beauty of the Yosemite "back country" feature the scenic views included in this film. (1 reel.)

HISTORICAL

Travelers' Toll.—Shows how the public pays for good roads; how the gasoline tax serves the same purpose as the turnpike toll. (3 reels.)

Wheels of Progress.—A pictorial tale of transportation in America from the day of the bicycle and the fancy turnout to modern methods of transportation. How the gasoline engine revolutionized American transportation and how the motor car and the truck are contributing to progress and prosperity in the United States. (1 reel.)

MISCELLANEOUS

Crossing the Great Salt Desert.—Difficulties overcome by engineers in building the Wendover Road, a short cut across the Great Salt Desert in Utah; unusual road engineering; mirages. Of general interest. (1 reel.)

Road Building in the United States.—Methods of building important types of highway in the United States, as seen by the Pan American Highway Commission on a tour of this country; topsoil, gravel, penetration macadam, concrete, and asphaltic concrete roads; how the work is financed. Of general interest. (2 reels.)

A Highway of Friendship.—Reception of the Pan American Highway Commission by Federal and State officials and people of States visited; interesting places and events in North Carolina, Kentucky, Ohio, Illinois, Minnesota, Wisconsin, Michigan, Pennsylvania, New Jersey, and the city of Washington. Of general interest. (2 reels.)

Roads for All America.—Visit to Buenos Aires, Argentina, of the United States delegation to the first Pan American Road Congress. The Pan American Highway commission with President Coolidge, the Panama Canal, types of highways, scenes of general interest, and leading men, including the Presidents of Panama, Peru, Chile, Argentina, Uruguay, and Brazil. Of general interest. (6 reels.)

SOUTH AMERICAN ROAD BUILDING

Highways of Argentina.—A sight-seeing tour of the wealthy capital of Buenos Aires over its broad avenues; an inspection trip of the rural road construction in the Provinces of Cordoba, Santa Fé, and Entre Rios. Of general interest. (1 reel.)

Highways of Brazil.—Traffic on the streets and drives of Rio de Janeiro; the high-type concrete pavement to Petropolis; the road to São Paulo, the coffee center; the Paineiras road; and typical road construction scenes. Of general interest. (2 reels.)

Highways of Chile.—An automobile trip from the seaport of Santiago to the capital at Valparaiso, annual review of the crack military and naval cadets and the transportation facilities of the rich copper and nitrate mines in northern Chile. Of general interest. (2 reels.)

Highways of Peru.—The remains of the early highway system laid out by the Incas, the modern highway system under construction following the plan of the Incas, and the great national obstacles to road construction. Of general interest. (2 reels.)

Highways of Uruguay.—The well-paved and lighted avenues of Montevideo, the roads radiating into the agricultural and cattle-raising regions, the new road under construction from Montevideo to Colonia, and characteristic scenes of the country. Of general interest. (1 reel.)

Highways of Venezuela.—A loop automobile trip from the harbor of La Guaira, on the northern coast of Venezuela, to the capital at Caracas, and through the fertile agricultural region in the vicinity of Maracay and Valencia to Puerto Cabello on the Caribbean Sea. Of general interest. (2 reels.)

Highway Glimpses of Colombia and the Dutch West Indies.—Street and rural road scenes in the vicinity of Puerto Colombia and Barranquilla in Colombia and traffic at Willemstad on the Island of Curaçao in the Caribbean Sea. Of general interest. (1 reel.)

Highway Glimpses of Panama.—Business activity of the streets of the new city of Panama contrasted with the ruins of Old Panama laid waste by the buccaneer and freebooter, Sir Henry Morgan, and the rural road development of the Republic. Of general interest. (2 reels.)

Highway Glimpses of Trinidad.—The superb highway system of the island, the natural bituminous deposit of Asphalt Lake, which is the source of material for many streets and roads in the United States, and typical road construction scenes. Of general interest. (1 reel.)

1930 MOTOR VEHICLE REGISTRATIONS AND RECEIPTS

Reported by L. A. ABBOT, Associate Statistical Engineer, United States Bureau of Public Roads

DURING the year 1930 there were 26,523,779 different motor vehicles regularly registered by the various States and the District of Columbia, according to reports received by the Bureau of Public Roads. The States and the District of Columbia collected in registration fees, licenses, operators' permits, fines, etc., gross receipts totaling $355,704,860.

If the above two figures for the year 1930 are compared with similar totals for 1929, it will be noted that there was a gain in registration of 22,336 motor vehicles, or only eight hundredths of 1 per cent, and a gain in receipts of $7,861,317, or 2.3 per cent.

The 10-year growth in registration is shown in Table 1, covering the years 1921 to 1930, inclusive. The first three columns show the generally used method of comparing each total to the previous year and basing the increase and percentage of increase on the preceding year regardless of whether or not it was a peak, average, or depressed year. The last three columns show the results obtained by using a constant basis (the year 1921). The last column shows that 1929, with more than 19 per cent increase over the preceding year, on the 1921 basis, was a remarkable year in motor vehicle expansion.

TABLE 1.—*Ten-year growth in motor-vehicle registration, from 1921 to 1930, inclusive, showing accumulated increase since 1921, with annual increases based on 1921*

Year	Total motor vehicles registered	Increase over previous year		Accumulated increase over 1921		Annual per cent increase based on 1921
		Number	Per cent	Number	Per cent	
1921	10,463,295					
1922	12,238,375	1,775,080	17.0	1,775,080	17.0	
1923	15,090,936	2,852,561	23.3	4,627,641	44.2	27.2
1924	17,593,677	2,502,741	16.6	7,130,382	68.1	23.9
1925	19,937,274	2,343,597	13.3	9,473,979	90.3	22.2
1926	22,001,393	2,064,119	10.4	11,538,098	110.2	19.9
1927	23,133,241	1,131,848	5.1	12,669,946	121.1	10.9
1928	24,493,124	1,359,883	5.9	14,029,829	134.1	13.0
1929	26,501,443	2,008,319	8.2	16,038,148	153.3	19.2
1930	26,523,779	22,336	.1	16,060,484	153.5	.2

In the table of motor-vehicle registration the total motor vehicles are classified into two groups according to type of service. One group for passenger service comprises passenger cars, taxis and busses, and the second group is made up of motor trucks and road tractors, which are used for freight service. From a comparison of these group totals for 1930 with the same groups in 1929, it is noted that the passenger vehicle registration decreased, while that of motor trucks increased 3 per cent. This percentage increase is only one-third of the 1929 motor truck increase, yet it indicates that business was moving ahead in spite of the depression. This indication relating to business is also confirmed by a noteworthy increase in trailers, which are not included in motor-vehicle registration; for in 1930, trailer registration increased 36 per cent over the previous year's figures.

Regarding the registration by States, New York still leads with 2,307,730 motor vehicles, gaining 44,471, and is followed closely by California with 2,041,356. This State made the largest net gain, namely 67,015. These two States are the only ones now registering more than 2,000,000 motor vehicles. There are five other States having over 1,000,000 registration as follows: Ohio, 1,759,363; Pennsylvania, 1,753,521; Illinois, 1,638,260; Texas, 1,365,896; and Michigan, 1,328,209. The order of precedence is the same as in 1929, except that Texas, with a gain of 17,789, has passed Michigan, which reported the greatest loss of any State, amounting to 66,893, or 4.8 per cent under 1929. This drop in Michigan, which accounts for almost one-quarter of the decrease in registration of the 21 States showing decreases, is due not only to the acute manufacturing depression in that State but probably also to a large scrapping of cars sold for junk to manufacturers desirous of ridding the market of old cars.

The increase in receipts already mentioned was chiefly due to higher registration fees, but gains in both trailer registrations and operators' permits helped to increase the total. In the disposition of the gross receipts according to the laws of the various States, the collection and administration expenses amounted to $19,196,926 (excluding these expenses in six States which make State appropriations); State highways were allocated $222,146,682, a little less than in 1929; local roads received $68,577,899, about 3 per cent increase; road bond payments were allotted $36,309,682 and $9,473,671 was reported "for other purposes." This latter classification includes $3,454,962 for State highway patrol; $1,403,280 for city streets, and for a free bridge commission (New Jersey); $185,587 assigned to funds used for refunds; and last, $4,429,842 (1¼ per cent of gross receipts) to general funds of city, county and State as noted on the table of motor vehicle receipts.

MOTOR VEHICLE REGISTRATIONS, 1930 [1]

[Compiled for calendar year from reports of State authorities]

State	1930-registered motor vehicles individually and commercially owned [2]			Other registered vehicles		Tax-exempt official motor cars and motor cycles		Number of licenses or permits		Total registered motor cars and trucks, in 1929	Year's change in motor-vehicle registrations		State
	Grand total registered motor cars and trucks	Passenger automobiles, taxis, and busses [3]	Motor trucks and road tractors	Trailers	Motor cycles	United States,[4] State, and local cars	Motor cycles (official)	Dealers	Operators and chauffeurs		Number increase, or decrease (−)	Per cent	
Alabama	277,146	239,170	37,976	3,550	663	1,368		4,181	2,989	285,533	−8,387	−2.9	Alabama.
Arizona	110,525	98,480	12,045	1,346	390	1,457	47	117	13,468	109,013	1,512	1.4	Arizona.
Arkansas	220,204	193,218	26,986	2,805	380	939		481	5,137	233,128	−12,924	−5.5	Arkansas.
California	2,041,356	1,810,969	230,387	48,532	9,405	18,122		3,325		1,974,341	67,015	3.4	California.
Colorado	308,509	276,847	31,662	188	1,059	283		3,474	7,296	303,489	5,020	1.7	Colorado.
Connecticut	331,026	279,830	51,196	480	2,371	2,390	277	3,178	409,557	328,063	2,963	.9	Connecticut.
Delaware	56,109	45,533	10,576	412	295	44		680	63,452	54,960	1,149	2.1	Delaware.
Florida	327,801	274,705	53,096	5,000	1,490	3,871	160	1,724	2,514	345,977	−18,176	−5.2	Florida.
Georgia	341,580	294,461	47,119	2,704	1,178	934		2,796	3,045	358,905	−17,325	−4.8	Georgia.
Idaho	119,077	104,526	14,551	450	359	1,329	11	441	923	118,074	1,003	.8	Idaho.
Illinois	1,638,260	1,430,676	207,584	7,341	6,245	979		4,368	108,538	1,615,088	23,172	1.4	Illinois.
Indiana	875,763	747,366	128,397	13,646	2,862	8,593		2,706	58,847	866,715	9,048	1.0	Indiana.
Iowa	778,386	706,196	72,190	1,542	1,712	3,789	77	2,340	17,911	784,450	−6,064	−.8	Iowa.
Kansas	594,523	511,384	[5] 83,139	1,083	1,275	2,833	40	2,607		581,223	13,300	2.3	Kansas.
Kentucky	331,002	295,161	35,841	[6]	681	2,282		1,082	12,481	332,848	−1,846	−.6	Kentucky.
Louisiana	275,283	230,586	44,697	5,443	518	209		407	22,735	280,868	−5,585	−2.0	Louisiana.
Maine	186,157	148,722	37,435	2,207	1,170	1,629	68	1,239	227,723	184,506	1,651	.9	Maine.
Maryland	321,702	283,870	37,832	1,008	1,941	2,000		5,788	82,468	319,873	1,829	.6	Maryland.
Massachusetts	846,206	743,288	102,918	701	4,642	1,556		3,112	982,795	817,704	28,502	3.5	Massachusetts.
Michigan	1,328,209	1,161,051	[5] 167,158	44,441	3,530	371		2,034	70,710	1,395,102	−66,893	−4.8	Michigan.
Minnesota	732,972	624,902	108,070	9,541	1,825	3,680		1,991		730,399	2,573	.4	Minnesota.
Mississippi	237,094	203,443	33,651	3,890	217	74		4,942		250,011	−12,917	−5.2	Mississippi.
Missouri	761,600	670,145	[5] 91,455	3,742	1,746	2,086	5	2,438	36,221	756,680	4,920	.6	Missouri.
Montana	135,168	109,549	25,619	56	242	1,640		575	412	140,387	−5,219	−3.7	Montana.
Nebraska	426,229	367,587	58,642	10,320	900	1,681		3,251		418,226	8,003	1.9	Nebraska.
Nevada	29,645	23,388	6,257	330	74	544		109		31,915	−2,270	−7.1	Nevada.
New Hampshire	112,183	93,155	19,028	935	1,132	484		600	130,023	108,880	3,303	3.0	New Hampshire.
New Jersey	852,850	719,696	133,154	2,639	5,998	8,420		3,207	1,018,335	832,332	20,518	2.5	New Jersey.
New Mexico	84,150	70,450	[7] 13,700	850	200	956	10	230		78,374	5,776	7.4	New Mexico.
New York	2,307,730	1,966,981	340,749	10,205	12,355	20,886	1,302	4,965	2,900,198	2,263,259	44,471	2.0	New York.
North Carolina	453,241	397,133	56,108	5,845	1,350	8,070		5,575		483,602	−30,361	−6.3	North Carolina.
North Dakota	183,019	155,383	27,636	[8]	235	3		650		188,046	−5,027	−2.7	North Dakota.
Ohio	1,759,363	1,555,093	204,270	24,356	6,886	13,854		3,772	60,925	1,766,614	−7,251	−.4	Ohio.
Oklahoma	550,331	490,947	59,384	[8]	1,226	5,230				570,791	−20,460	−3.6	Oklahoma.
Oregon	252,123	234,766	17,357	1,893	1,348	3,003	58	517	26,705	269,007	−16,884	−6.3	Oregon.
Pennsylvania	1,753,521	1,534,834	218,687	5,684	12,961	2,478		10,704	2,181,006	1,733,283	20,238	1.2	Pennsylvania.
Rhode Island	136,423	116,792	19,631	90	770	901	112	333	156,532	134,009	2,414	1.8	Rhode Island.
South Carolina	218,402	192,141	26,261	2,976	559	3,550		1,259	314,702	231,274	−12,872	−5.6	South Carolina.
South Dakota	205,172	180,195	24,977	[6]	239	997		894		204,199	973	.5	South Dakota.
Tennessee	368,259	330,436	37,823	[6]	1,228	4,552		628		362,431	5,828	1.6	Tennessee.
Texas	1,365,896	1,159,139	206,757	29,853	4,045	2,505		3,879	18,804	1,348,107	17,789	1.3	Texas.
Utah	113,997	96,128	17,869	[6]	488	1,373		300	2,650	112,661	1,336	1.2	Utah.
Vermont	86,624	78,398	8,226	315	524	28		363	97,699	93,030	−6,406	−6.9	Vermont.
Virginia	375,889	318,582	57,307	1,318	2,084	5,053		4,063	9,962	387,205	−11,316	−2.9	Virginia.
Washington	446,062	382,874	63,188	3,147	1,993	6,449	179	4,738	85,714	442,341	3,721	.8	Washington.
West Virginia	266,273	225,900	40,373	928	1,330	3,057	123	10,702	87,606	268,888	−2,615	−1.0	West Virginia.
Wisconsin	782,562	677,452	105,110	715	2,666	5,225	324	2,912	79,059	793,502	−10,940	−1.4	Wisconsin.
Wyoming	61,501	51,579	9,922	[6]	121	591		352		60,680	821	1.4	Wyoming.
District of Columbia	156,676	139,733	16,943	[8]	903	3,412	153	1,759	71,743	151,450	5,226	3.4	District of Columbia.
Total	26,523,779	23,042,840	3,480,939	262,507	107,811	[9] 173,619	2,946	121,788	9,370,885	26,501,443	22,336	.08	Total.

[1] This table lists only the number of registrations, licenses and permits. For financial statement see table on next page.

[2] The first 3 columns show regularly registered motor cars and trucks, eliminating reregistrations, nonresident registrations, etc.

[3] Busses are included with passenger cars except as noted in next column. A bus census is published in a magazine devoted to busses.

[4] These official cars are exempted from paying regular registration fees and are excluded from "registered motor vehicles."

[5] Busses included with trucks.

[6] Classified with trucks.

[7] Large increase in trucks due to change in recording small commercial trucks, formerly classified with passenger cars.

[8] Not registered.

[9] Includes 7,859 "cars-at-large," owned by United States Government and not allocated to any State

MOTOR VEHICLE REGISTRATION FEES, LICENSES, PERMITS, FINES, ETC., 1930 [1]

[Compiled for calendar year from reports of State authorities]

State [3]	Registration receipts [4]					Miscellaneous receipts [2]			Disposition of gross receipts					State [3]
	Total gross receipts	Motor-car receipts		Other vehicles		Dealers' licenses	Chauffeur and operator permits	Other miscellaneous	For highway purposes					
		Total motor cars	Passenger cars and busses	Trucks and tractors	Trailers	Motor cycles			Collection and administration [3]	State highways	Local roads	State and county road bonds [4]	For other purposes	
Alabama [2]	$3,799,761								$225,541	$890,029	$740,953	$1,943,238		Alabama.
Arizona	734,626	$540,516	$313,233	$227,283	$13,438	$1,186	$890	$13,682	$164,914	734,626				Arizona.
Arkansas	4,283,959	4,242,757					5,746	35,456	69,135	2,874,910		[5] 1,339,914		Arkansas.
California	9,858,810	8,639,234	6,202,593	2,436,641	313,170	33,612	47,692	183,701	641,401	1,493,292	3,387,537	3,387,537	[6] $1,590,441	California.
Colorado	1,901,230	1,784,685	1,415,926	368,759	2,558	1,832	28,204	14,535	69,416	131,723	884,754	884,753		Colorado.
Connecticut	8,290,404	6,353,650	4,878,046	1,475,604	17,741	9,725	93,628	1,234,969	580,691	716,337	7,574,067			Connecticut.
Delaware	1,110,047	864,085	636,215	227,870	4,501	1,047	7,710	192,229	40,475		598,292	[7] 511,755		Delaware
Florida	4,813,495	4,728,559	3,545,445	1,183,114	24,358	5,435	22,698	4,757	27,688	288,810	3,393,539	1,131,146		Florida.
Georgia	4,482,257	4,417,915	3,605,889	812,026	21,051	5,231	31,087	6,032	941	211,212	4,271,045			Georgia.
Idaho	1,998,290	1,958,605	1,548,966	409,639		1,803	26,190	1,846	9,846	59,949	193,834	1,744,507		Idaho.
Illinois	18,444,247	17,292,244	12,858,506	4,433,738	151,722	20,138	84,040	364,316	531,787		10,468,503	7,975,744		Illinois.
Indiana	6,346,879	5,941,606	4,532,881	1,408,725	65,352	5,056	54,320	15,327	265,218	326,834	6,020,045			Indiana.
Iowa	12,693,621	11,978,356	10,251,169	1,727,187	16,994	6,066	77,815	35,823	578,567	612,753	11,955,810		[8] 125,058	Iowa.
Kansas [2]	6,084,348									170,735	4,163,613	1,750,000		Kansas.[2]
Kentucky	5,547,069	5,375,879	4,204,514	1,171,365	[9]	5,253	30,290	33,594	102,053	275,612	4,685,774	585,683		Kentucky.
Louisiana	4,609,042	4,528,346						68,205	12,491	50,000	4,559,042			Louisiana.
Maine	3,166,642	2,423,428	1,815,083	608,345	7,273	6,162	48,505	510,410	170,864	316,422	1,443,300		1,266,922 [10] 139,998	Maine.
Maryland	3,437,796	2,712,922	2,303,398	409,524	26,341	9,354	30,456	212,717	446,006	343,780	2,475,213		[11] 618,803	Maryland.
Massachusetts	7,120,583	4,117,530	2,883,475	1,234,055	33,169	6,601	9,300	1,965,590	988,393	1,643,710	4,841,249		635,624	Massachusetts.
Michigan	22,482,412	20,771,694	15,998,470	4,773,224	371,124	13,794	91,148	226,896	1,007,756	875,279	14,525,073	6,000,000	1,082,060	Michigan.
Minnesota	11,062,150	10,926,205	8,936,703	1,989,502	35,125	6,662	36,864	30,083	27,211		6,977,428		[12] 4,053,439 [13] 31,283	Minnesota.
Mississippi [2]	3,046,393									159,818	241,324	2,645,251		Mississippi.[2]
Missouri [2]	10,150,000									498,517	6,078,983		3,572,500	Missouri.[2]
Montana	1,583,276	1,503,003	1,208,904	294,099	280	970	20,285	824	57,914	56,100		1,527,176		Montana.
Nebraska	3,804,950	3,631,476	3,117,510	513,966	10,944	2,911	44,131		115,488	116,600	1,106,505	2,581,845		Nebraska.
Nevada	373,966	287,489	206,365	81,124	2,245	330	3,527		80,375	14,069	189,647		170,250	Nevada.
New Hampshire	2,290,435	1,868,051				5,965	28,809	303,517	84,093	175,319	2,115,116			New Hampshire.
New Jersey	15,382,456	11,270,251	7,240,775	4,029,476	100,975	11,996	79,995	3,060,836	858,403	1,034,187	9,362,269	4,735,000	[14] 251,000	New Jersey.
New Mexico [2]	1,279,623									78,063	444,577	204,265	[15] 552,718	New Mexico.[2]
New York	40,857,715	36,459,709	26,545,479	9,914,230	167,608	54,241	211,171	3,026,794	938,192	2,662,798	28,260,942	6,088,134	[16] 3,845,841	New York.
North Carolina [2]	6,835,743									300,000	2,847,402		3,688,341	North Carolina.[2]
North Dakota	1,958,662	1,945,936	1,518,362	427,574		1,355	11,064		307	77,115	1,098,674	782,873		North Dakota.
Ohio	13,287,352	12,700,300			[17]	17,215	75,440	182,737	311,660	507,329	6,383,546	6,396,477		Ohio.
Oklahoma [2]	6,536,361									256,412	2,358,132	3,921,817		Oklahoma.[2]
Oregon	9,617,930	9,059,567	7,511,493	1,548,074	42,112	8,306	37,025	66,184	404,736	555,627	2,295,500	3,315,488	3,451,315	Oregon.
Pennsylvania	33,112,371	23,859,639	16,109,236	7,750,403	77,038	31,988	264,060	4,593,592	4,286,054	2,048,435	26,513,678		3,449,847 [10] 1,100,411	Pennsylvania.
Rhode Island	2,280,849	1,834,238	1,350,203	484,035	2,076	3,514	16,890	323,260	100,871	249,437	2,014,012	17,400		Rhode Island.
South Carolina	2,878,352	2,581,458	2,130,068	451,390	34,348	1,759	30,838	143,482	86,467	40,692	2,681,185		[18] 156,475	South Carolina.
South Dakota	2,959,913	2,900,648	2,462,517	438,131		856	22,010		36,399	92,574	1,462,557	1,404,782		South Dakota.
Tennessee [2]	4,767,239									88,830	2,220,656	2,220,656	[10] 237,097	Tennessee.[2]
Texas	13,961,362	13,061,103	9,252,280	3,808,823	289,832	17,662	57,683	56,349	478,733	843,749	4,153,585	8,964,028		Texas.
Utah [2]	855,584									118,084	402,100		335,400	Utah.[2]
Vermont	2,392,152	2,058,025	1,679,680	378,345	[9]	3,512	28,343	252,077	50,195		2,392,152			Vermont.
Virginia	6,493,989	6,234,559	5,108,796	1,125,763	11,508	7,920	67,581	54,726	117,695	212,206	6,116,058		[10] 165,725	Virginia.
Washington	7,616,676	7,354,120	5,744,588	1,609,532	65,539	11,089	85,735	87,799	12,394	187,519	5,279,245	2,062,113	[10] 87,799	Washington.
West Virginia	4,702,812	4,295,618	3,331,457	964,161	5,514	5,869	44,766	141,127	209,918	209,798	1,622,139		2,833,333 [8] 37,542	West Virginia.
Wisconsin	13,083,521	12,724,232	9,123,136	3,601,096	41,089	15,415	101,078	22,197	179,510	700,000	6,897,506	5,486,015		Wisconsin.
Wyoming [2]	691,509										691,509			Wyoming.[2]
District of Columbia	636,001	178,907	152,257	26,650		918	1,759	215,229	239,188	102,524			[19] 533,477	District of Columbia.
Detailed totals	311,658,299	275,406,545	[20]	[20]	1,955,025	342,748	1,958,773	17,680,898	14,314,310					Detailed totals.
Grand total	355,704,860									19,196,926	222,146,682	68,577,899	36,309,682	9,473,671	Grand total.

[1] Financial date only on this table. For number of registrations, etc., see table on preceding page.
[2] Ten States thus noted do not report complete details, and receipts are not included in "detailed totals."
[3] States for which no figures are shown make State appropriations for administration.
[4] Payments on State bonds except as noted.
[5] Includes payments on county bonds $1,026,754.
[6] Includes $10,495 for refunds; remainder State highway patrol.
[7] Includes payments on county bonds, $362,568.
[8] For refunds.
[9] Included with truck receipts.
[10] For State highway patrol.
[11] For Baltimore Streets.
[12] For payments on county bonds.
[13] For State general fund.
[14] For free bridge commission.
[15] For State general fund $252,328; remainder to general county fund.
[16] For New York City general fund.
[17] Included with motor car receipts.
[18] For refunds $12,492; remainder State highway patrol.
[19] For signals and streets as appropriated by Congress.
[20] Only 34 States and District of Columbia separate motor-car receipts as follows: Passenger car and bus receipts $189,723,618, and truck and tractor receipts $62,343,473.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.
Report of the Chief of the Bureau of Public Roads, 1929.
Report of the Chief of the Bureau of Public Roads, 1930.

DEPARTMENT BULLETINS

No *136D. Highway Bonds. 20c.
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*660D. Highway Cost Keeping. 10c.
*691D. Typical Specifications for Bituminous Road Materials. 10c.
1216D. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
1279D. Rural Highway Mileage, Income, and Expenditures 1921 and 1922.
1486D. Highway Bridge Location.

DEPARTMENT CIRCULARS

No 331C. Standard Specifications for Corrugated Metal Pipe Culverts.

TECHNICAL BULLETIN

No 55T. Highway Bridge Surveys.

MISCELLANEOUS CIRCULARS

No. 62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-Aid Highway Projects.
*93M. Direct Production Costs of Broken Stone. 25c.
109M. Federal Legislation and Regulations Relating to the Improvement of Federal-Aid Roads and National-Forest Roads and Trails, Flood Relief, and Miscellaneous Matters.

MISCELLANEOUS PUBLICATIONS

No. 76MP. The Results of Physical Tests of Road-Building Rock.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *914Y. Highways and Highway Transportation. 25c.
937Y. Miscellaneous Agricultural Statistics.
1036Y. Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio. (1927)
Report of a Survey of Transportation on the State Highways of Vermont. (1927)
Report of a Survey of Transportation on the State Highways of New Hampshire. (1927)
Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio. (1928)
Report of a Survey of Transportation on the State Highways of Pennsylvania. (1928)

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

FEBRUARY 28,1931

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage	MILEAGE Total	Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage	MILEAGE Total		
Alabama	2,210.7	$ 4,521,782.51	$ 2,191,003.00	180.4	15.8	196.2	$ 1,958,356.12	$ 907,465.28	27.0	68.6	86.6	$ 4,058,209.08	Alabama
Arizona	886.7	4,119,489.04	3,183,956.66	145.9	162.2	308.1	774,768.25	425,227.35	19.0	40.2	59.2	1,566,028.21	Arizona
Arkansas	1,811.6	7,030,511.62	3,244,190.29	205.6	19.5	225.1	2,099,376.14	999,655.11	29.4	45.1	74.5	1,199,901.42	Arkansas
California	1,949.0	8,078,426.67	3,538,178.46	179.7	31.8	211.5	1,477,097.26	420,657.14	49.8	2.3	62.1	3,852,608.09	California
Colorado	1,287.1	4,307,101.13	2,315,285.04	163.5	67.7	231.2	950,847.25	498,490.32	40.8	21.3	61.9	3,257,731.19	Colorado
Connecticut	258.2	1,524,709.41	661,789.48	7.6		7.6	2,384,628.77	816,706.04	27.0		27.0	504,913.04	Connecticut
Delaware	318.9	105,632.85	63,316.42	4.4		4.4	1,257,177.78	516,724.26	51.8		61.8	134,468.17	Delaware
Florida	527.1	4,517,261.88	2,187,119.82	103.4		103.4	949,280.26	444,589.36	46.7		46.7	2,260,870.67	Florida
Georgia	2,712.0	7,015,390.64	3,346,843.00	193.9	109.5	303.4	4,014,721.48	1,872,244.99	140.0	44.3	184.3	2,414,673.97	Georgia
Idaho	1,297.3	2,232,519.81	1,355,810.82	155.3	11.3	166.6	824,004.42	465,027.87	64.7	7.3	72.0	1,460,117.22	Idaho
Illinois	2,183.4	21,248,092.12	9,595,697.70	590.4	57.3	647.7	4,391,959.60	2,023,452.74	133.9	2.2	136.1	5,268,861.49	Illinois
Indiana	1,579.0	3,838,237.47	1,429,853.80	83.6		83.6	4,664,043.40	2,059,612.65	141.1		141.1	3,228,024.04	Indiana
Iowa	3,225.6	105,290.11	1,892.77		4.1	4.1	7,713,655.16	3,333,554.28	157.9	89.4	247.3	24,088.04	Iowa
Kansas	2,991.7	6,146,363.84	2,951,872.20	357.4	63.8	421.2	1,409,202.77	671,342.92	115.8	16.3	132.1	2,744,811.20	Kansas
Kentucky	1,565.4	4,750,196.33	2,116,694.66	165.9	71.4	237.3	1,245,189.90	672,306.31	62.2	22.0	84.2	1,302,992.39	Kentucky
Louisiana	1,429.2	4,494,662.36	2,216,546.33	191.3	13.8	205.1	4,314,026.10	1,933,180.17	87.6	10.0	97.6	413,902.19	Louisiana
Maine	691.3	2,000,256.80	654,021.27	43.5		43.5	1,572,554.78	620,895.13	26.8		26.6	1,568,538.42	Maine
Maryland	706.5	125,448.40	38,825.00		1.5	1.5	734,444.38	329,911.42	19.7	2.4	22.1	711,369.97	Maryland
Massachusetts	710.7	8,439,645.87	2,123,067.27	73.2		73.2	73,542.42	22,980.00	1.6		1.6	2,613,059.23	Massachusetts
Michigan	1,777.1	9,332,000.02	3,858,681.53	231.8	34.2	266.0	889,076.01	408,377.17	49.3	1.5	50.8	3,680,601.12	Michigan
Minnesota	4,313.4	2,041,829.15	753,117.07	22.3	92.9	115.2	5,151,384.55	1,987,745.64	64.7	169.3	214.0	1,040,956.03	Minnesota
Mississippi	1,806.6	1,571,632.45	761,989.35	39.1	42.2	81.3	1,285,450.34	636,393.64	43.6	23.1	86.7	4,599,176.39	Mississippi
Missouri	2,639.2	6,790,285.18	2,499,676.93	131.6	37.5	169.0	2,570,650.93	1,147,299.89	85.8	26.0	112.8	2,299,870.20	Missouri
Montana	1,885.6	7,880,704.86	4,441,885.64	621.4	40.9	662.3	3,476,591.22	1,957,561.95	232.8	60.4	283.2	3,019,201.27	Montana
Nebraska	3,874.4	4,691,609.56	2,250,879.60	264.9	66.8	320.7	1,631,092.60	703,606.84	40.8	77.6	118.4	3,663,569.91	Nebraska
Nevada	1,277.4	1,005,734.41	871,720.23	52.4	63.4	116.8	546,593.67	240,300.21	10.2	49.0	59.2	1,665,642.06	Nevada
New Hampshire	392.1	609,439.04	229,098.40	9.7		9.7						555,223.93	New Hampshire
New Jersey	554.9	3,577,523.26	1,119,784.41	32.4		32.4	1,585,049.24	634,539.13	24.2		24.2	1,563,079.54	New Jersey
New Mexico	2,023.6	4,291,546.06	2,880,595.50	180.7	25.5	206.2	107,277.63	67,700.35	.3		.3	1,938,697.16	New Mexico
New York	2,674.8	15,432,610.33	3,189,755.00	215.7		215.7	19,863,626.90	7,989,742.50	344.1		344.1	5,238,397.91	New York
North Carolina	1,893.6	4,426,206.29	2,074,275.97	139.5	33.6	173.0	1,529,568.50	757,717.43	84.8	10.2	95.0	3,320,318.82	North Carolina
North Dakota	4,693.0	1,695,242.85	847,437.09	224.8	128.2	353.0	1,651,754.61	824,675.28	236.3	168.3	404.6	2,335,458.81	North Dakota
Ohio	2,541.7	11,604,392.49	3,583,280.18	162.5	12.0	174.5	2,963,355.50	900,389.68	35.4	17.1	52.5	3,929,614.49	Ohio
Oklahoma	1,989.1	4,433,224.42	2,081,041.74	146.3	64.1	209.4	2,762,245.55	1,519,724.66	180.9	16.6	197.4	1,291,583.00	Oklahoma
Oregon	1,264.6	6,410,149.02	3,647,017.27	200.3	78.7	278.0	1,014,346.85	452,465.61	56.7	11.9	68.6	1,091,986.95	Oregon
Pennsylvania	2,658.1	7,304,923.48	2,600,346.48	47.0		47.0	404,741.83	69,187.89	2.9		2.9	6,157,112.91	Pennsylvania
Rhode Island	209.6	1,720,027.14	664,901.84	26.4		26.4	181,782.06	90,891.03	3.7		3.7	546,776.15	Rhode Island
South Carolina	1,819.9	7,027,073.48	3,008,001.03	100.7	178.2	278.9	1,335,214.88	685,373.98	.1	67.1	57.2	102,336.51	South Carolina
South Dakota	3,773.4	3,242,704.64	1,791,413.91	299.0	75.4	374.4	870,191.12	478,604.96	118.3	61.3	159.6	2,056,320.20	South Dakota
Tennessee	1,448.6	2,230,944.97	1,114,232.84	104.6		104.6	1,994,965.00	990,473.68	78.4		78.4	2,480,085.80	Tennessee
Texas	7,036.7	12,746,853.18	6,456,814.53	634.1	107.9	742.0	4,876,976.25	2,308,820.33	130.0	191.6	321.6	7,392,332.91	Texas
Utah	985.8	1,410,125.09	1,027,795.86	94.6	68.6	153.2	815,124.10	278,686.70	49.6	40.6	90.0	1,336,177.19	Utah
Vermont	303.6	269,217.03	111,090.83	3.1		3.1	861,308.36	340,907.17	22.6		22.6	235,909.27	Vermont
Virginia	1,528.5	4,590,875.76	2,221,631.43	182.5	16.4	198.9	884,463.18	328,587.89	28.5	14.1	42.6	1,559,972.14	Virginia
Washington	1,012.2	3,229,842.46	1,484,350.00	76.9	36.3	113.2	702,010.88	306,386.24	21.1		21.1	2,121,925.32	Washington
West Virginia	766.5	4,294,672.68	1,632,085.09	99.1	12.2	111.3	1,467,855.03	600,207.34	31.4	.3	31.7	686,634.56	West Virginia
Wisconsin	2,490.4	3,496,577.92	1,492,374.66	81.8	13.0	94.8	382,045.63	186,121.15	24.2		24.2	2,420,775.29	Wisconsin
Wyoming	1,729.6	2,963,986.87	1,937,053.56	188.8	125.2	314.0	454,805.19	269,465.64	10.0	70.8	80.9	1,034,513.72	Wyoming
Hawaii	41.4	1,081,873.35	473,002.56	31.6		31.6	452,952.21	126,119.93	7.6		7.6	1,711,971.90	Hawaii
TOTALS	89,642.5	237,094,945.27	103,330,294.21	7,489.4	1,961.8	9,451.2	105,531,389.04	46,100,795.31	3,249.6	1,399.0	4,648.5	108,360,973.58	TOTALS

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

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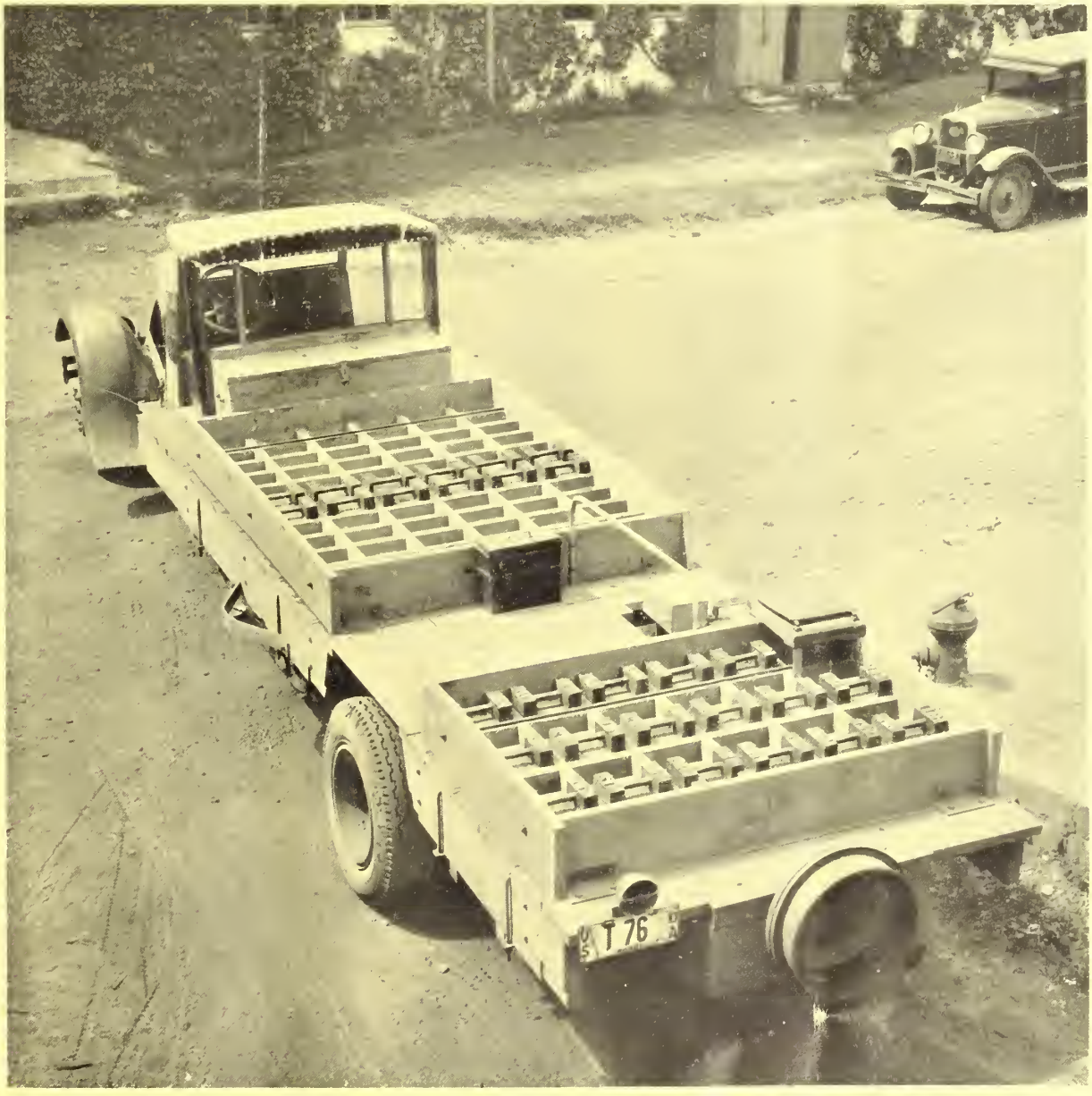
PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 12, NO. 2 APRIL, 1931



BUS CHASSIS USED IN IMPACT TESTS

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

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IMPACT REACTIONS DEVELOPED BY A MODERN MOTOR BUS

Reported by JAMES A. BUCHANAN, Associate Engineer of Tests, U. S. Bureau of Public Roads

ONE of the most important trends in the use of highways to-day is that toward higher vehicle speeds, particularly with the heavier wheel loads. At the present time, this trend is most evident in motor bus operations, although fast-freight highway transportation is a rapidly developing industry. This condition of heavy wheel loads operating at sustained high speeds has been made practicable by the comparatively recent introduction of high-pressure and balloon tires adequate for and economical in such service. A study of the impact reactions produced on pavements by vehicle wheels under such intensive operating conditions was the object of a research investigation recently completed by the Bureau of Public Roads with the cooperation of the Rubber Manufacturers' Association and the Society of Automotive Engineers.

In arranging a schedule of tests to be followed several factors relating to equipment and scope of conditions were considered. The vehicle and tire equipment should be representative of modern high-speed traffic. The loads and speeds should cover an adequate range in operating conditions. The roughness conditions should involve both artificial and natural obstructions, the artificial obstructions bringing out the influences of various factors on impact reactions and the natural obstructions indicating the actual forces developed under operating conditions.

Figure 1 gives a general view of the vehicle used. It is a modern high-speed bus chassis on which a special body was built to meet the needs of the tests. The floor of this body was divided into a number of small compartments, as shown on the cover page, for retaining 100-pound cast-iron weights used to vary the load conditions. The wheel base is 233 inches. Each rear spring is provided with overload booster leaves which operate in two stages. In order to provide clearance between the rear spring and the large dual tires tested, it was necessary to apply a spacer ring on each rear hub between the flange and the inner wheel disc. These 1½-inch wide spacers remained in place throughout the tests so that all tires would be tested under the same conditions. This proved to be an unnecessary precaution, as accelerations obtained with 7 and 9.00 inch dual tires before the spacers were applied were the same as those obtained afterwards.

FIGURE 1.—GENERAL VIEW OF BUS USED IN IMPACT TESTS

TIRE EQUIPMENT SELECTED TO COVER TYPES IN ACTUAL USE

The tire equipment tested included both the high-pressure pneumatic and the low-pressure pneumatic or balloon types. The range in section size was from 7-inch high-pressure to 12.00-inch low-pressure and inflations varied from 107 to 53 pounds per square inch. Two rims diameters were used—20 and 24 inch.

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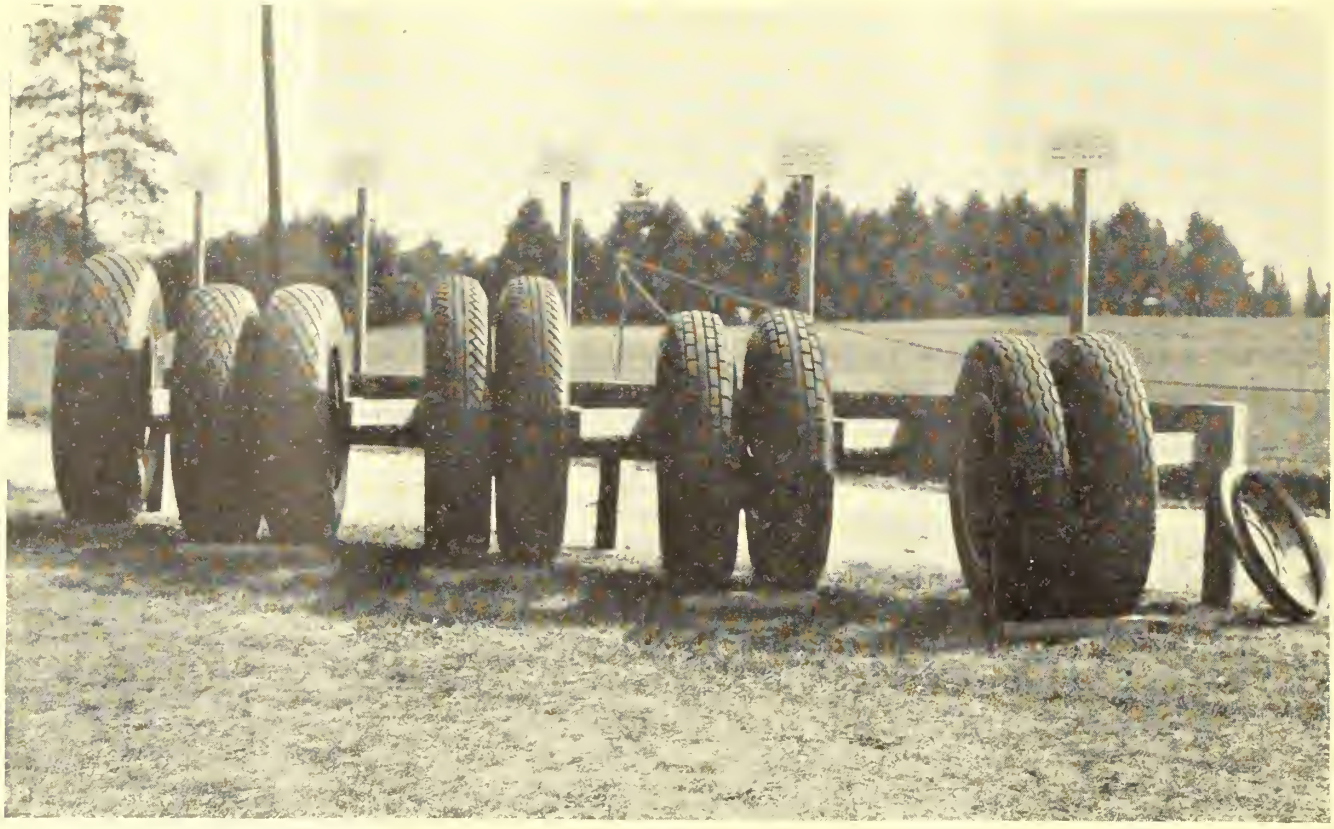

FIGURE 2.—SOME OF THE TIRE EQUIPMENT USED IN THE BUS TESTS

The tires were mounted directly on appropriate disc wheels prior to the tests, and tire changes were quickly effected by changing these disc wheels, which were bolted on the hub centers of the bus chassis.

A general view of the range in tire sizes is given in Figure 2. The tires were the products of four manufactures, but all had conventional tread designs used in modern commercial and bus service.

A complete list of the tire sizes tested is given in Table 1, together with the inflation pressure and load conditions selected for each tire. In determining these pressures and loads, three factors were considered. First, it was desirable to make comparisons on an equal load basis. Second, it was desirable to include tests conforming as closely as possible with recommended standard practice. Third, it was desirable to include test conditions varying from recommended practice for at least a part of the range found in actual operating conditions. The recommended or standard inflation pressures for various loads on balloon and high-pressure tires were obtained from the Yearbook of the Tire and Rim Association, published April, 1930, and are given in Tables 2 and 3. In this connection it is interesting to note that the actual practice of eight fleet operators using more than 5,000 buses was ascertained through a survey recently conducted by the National Association of Motor Bus Operators and was found to range from recommended inflation pressures and loads to rather severe overload and under-inflation conditions.

The various tire and rim sizes, loads and inflation pressures, as shown in Table 1, provided for the determination of the influence of such factors as inflation pressure, load, tire type, tire section, rim diameter, rim width, and single versus dual mounting. The wheel loads and other data for the various combinations are given in Table 4.

TABLE 1.—*Tire, rim, load, and inflation conditions*

Tire section	Rim diameter	Rim width	Tire plies	Dual spacing	Inflation pressure used with—					
					3,000-pound load per tire	3,500-pound load per tire	4,000-pound load per tire	4,700-pound load per tire	6,000-pound load per tire	8,000-pound load per tire
Inches	*Inches*	*Inches*		*Inches*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
7	20	7	10	10½	107, 100, 93, 86	100	100	100	.....	.....
7	24	7	10	10½	93	.....	.....	.....	.....	.....
9.00	20	7	10	10½	65, 61, 57, 53	65	65	65,75	.....	.....
9.00	24	7	10	10½	54	63	.....	.....	.....	.....
9.00	20	8	10	12	61	.....	.....	.....	.....	.....
9.75	20	8	12	12	55	.....	71	80	.....	.....
10.50	20	9–10	12	12¾	.....	.....	63	75	.....	.....
[1] 12.00	20	11	14	(Single)	.....	.....	.....	.....	82	95

[1] The single 12.00 by 20 inch balloon tire at an inflation pressure of 82 pounds per square inch has the same rated carrying capacity as the dual 9.00 by 20 inch balloon tires at 61 pounds per square inch and the 34 by 7 inch high pressure tires at 107 pounds per square inch.

EFFECTS OF ARTIFICIAL OBSTRUCTIONS AND NATURAL ROAD CONDITIONS STUDIED

Artificial obstructions creating roughness conditions on an otherwise smooth pavement surface were used in making tests for the determination of the influence of the various operating factors, because such roughness conditions could be made to a uniform section having sufficient width to obviate any uncertainty regarding the contact relation between the tires and the obstruction.

Through the courtesy of the Ordnance Department, a concrete road at the Aberdeen Proving Ground, Md., was made available. This road is level throughout its length and traffic on it was controlled so that there was no interference during test runs. Steel obstruc-

TABLE 2.—*Truck and bus balloon tire load and inflation table.*[1] *Minimum inflation pressure in pounds per square inch and corresponding load capacity in pounds*

Minimum inflation pressure	Tire section and rim diameter (inches)																Minimum inflation pressure
	5.50	6.00	6.50	7.00	7.50			8.25			9.00			9.75			
	20	20	20	20	20	22	24	20	22	24	20	22	24	20	22	24	
35	1,125																35
40	**1,225**	1,300															40
45		**1,400**	1,525														45
50			**1,650**	1,775	1,925	2,075	2,225	2,100	2,350	2,500							50
55				**1,900**	**2,100**	**2,250**	**2,400**	2,325	2,575	2,725	2,650	2,900	3,050				55
60								**2,550**	**2,800**	**2,950**	2,950	3,200	3,350	3,300	3,600	3,800	60
65											3,250	3,500	**3,650**	3,600	3,900	4,100	65
70														**3,900**	**4,200**	4,400	70

Minimum inflation pressure	Tire section and rim diameter (inches)															Minimum inflation pressure
	10.50			11.25			12.00			12.75			13.50			
	20	22	24	20	22	24	20	22	24	20	22	24	20	22	24	
65	4,100	4,400	4,600													65
70	4,400	4,700	4,900	4,750	5,100											70
75	**4,700**	**5,000**	**5,200**	5,100	5,450	5,700	5,450	5,900	6,150							75
80				**5,450**	**5,800**	**6,050**	5,850	6,300	6,550	6,400	6,900	7,200				80
85							**6,250**	**6,700**	**6,950**	6,800	7,300	7,600	7,200	7,800	8,100	85
90										7,200	7,700	8,000	7,700	8,300	8,600	90
95													8,200	8,800	9,100	95

[1] Tire and Rim Association standard, adopted Feb. 14, 1930. Bold-face type indicates maximum recommended loads. Duals will carry twice the load of corresponding singles.

TABLE 3.—*Truck and bus high-pressure tire load and inflation table.*[1] *Minimum inflation pressure in pounds per square inch and corresponding load capacity in pounds*

Minimum inflation pressure	Tire section and rim diameter (inches)														Minimum inflation pressure
	3½	4	4½	5		6		7		8		9		10		
				20	24	20	24	20	24	20	24	20	24	20	24	
50	600															50
55	675	800														55
60	**750**	900	1,050													60
65		**1,000**	1,150	1,325	1,575											65
70			**1,250**	1,450	1,700	1,700										70
75				[2] 1,575	1,825	1,825	2,125									75
80				**1,700**	**1,950**	[3] 1,950	2,250	2,200	2,600							80
85						2,075	2,375	2,350	2,750	2,775	3,125					85
90						**2,200**	**2,500**	2,500	2,900	2,900	3,300	3,300	3,800			90
100								**2,800**	**3,200**	3,250	3,650	3,700	4,200	4,150	4,650	100
110										**3,600**	**4,000**	4,100	4,600	4,600	5,100	110
120												**4,500**	**5,000**	5,050	5,550	120
130														**5,500**	**6,000**	130

[1] Tire and Rim Association standard, adopted Jan. 12, 1927. Bold-face type indicates maximum recommended loads. Duals will carry twice the load of corresponding singles.
[2] Maximum load for 6-ply tire only.
[3] Maximum load for 8-ply tire only.

tions 200 feet apart were located on this road at the end of an 0.8 mile straightaway and were secured in place by means of countersunk screws to nuts set in the pavement, flush with the surface.

Two obstructions were used, each 1½ inches in height. One was an inclined plane 30 inches long and the other was a rectangular obstruction 12 inches wide. These obstructions are shown in Figure 3. While such obstructions are somewhat more severe than those ordinarily encountered on a well maintained pavement carrying high-speed traffic, their use in these tests was necessary in order to bring out definitely the differences between the various factors being studied. Smaller obstructions, corresponding to those used in earlier motor truck impact tests [1] were tried but it

[1] These obstructions are described in an article entitled "Motor Truck Impact as Affected by Tires, Other Truck Factors and Road Roughness," Public Roads, Vol. 7, No. 4, June, 1926.

was found that the tire equipments used in these tests caused such low values of acceleration when small obstructions were encountered that differences in test conditions involving only minor changes in the factors being studied were likely to be obscured because of the dispersion due to unavoidable experimental errors.

The artificial obstructions used caused three types of impact reaction which are: (1) The drop of the wheel on the pavement after being raised by an inclined plane, (2) the shock of the wheel at striking a sharp elevation in the road, and (3) the drop of the wheel on the pavement following the shock condition just described. These obstruction conditions simulate heaved and raised joints in actual surface roughness conditions. There is probably little real difference in the action under either of the drop conditions described above other than variations in the height from which the wheel falls. In either case, the tire reacts against a plane surface. In the case of the shock reaction, there is a tendency for the obstruction to penetrate the tire or for the tire to envelop the obstruction and an entirely different situation exists.

1½ BY 30 INCH INCLINED PLANE

1½ BY 12 INCH RECTANGULAR BLOCK

FIGURE 3.—ROAD ROUGHNESS CREATED BY ARTIFICIAL OBSTRUCTIONS

TABLE 4.—*Wheel loads and other bus characteristics*

Tire size [1]	Weight wheel and tire	Unsprung weight	Unsprung mass	Load A		Load B		Load C		Load D	
				Sprung	Total	Sprung	Total	Sprung	Total	Sprung	Total
	Lbs.	*Lbs.*		*Lbs.*	*Lbs.*	*Lbs.*	*Lbs.*	*Lbs.*	*Lbs.*	*Lbs.*	*Lbs.*
34 x 7	350	1,974	61.3	4,060	6,034	5,080	7,054	6,075	8,049	7,525	9,499
38 x 7	394	2,018	62.7	4,060	6,078						
9.00–20/7	380	2,004	62.2	4,060	6,064	5,080	7,084	6,075	8,079	7,525	9,529
9.00–24	422	2,046	63.5	4,060	6,106	5,080	7,126				
9.00–20/8	396	2,020	62.7	4,060	6,080						
9.75–20	458	2,082	64.7	4,060	6,142			6,075	8,157	7,525	9,607
10.50–20	531	2,155	66.9					6,075	8,230	7,525	9,680
12.00–20	335	1,959	60.8	4,060	6,019			6,075	8,034		

Tare weights (including driver, operator, instrument, and hub spacers, with 34 by 7 inch original tire equipment):

	Pounds
Front axle	5,720
Rear axle	7,649
Total	13,369

	Pounds
Unsprung weight per rear wheel	1,962
Weight of 34 by 7 dual tires and wheel (original equipment)	338
Unsprung weight without tires and wheels	1,624

Front axle loads:	Pounds	Front wheel loads:	Pounds
Load A	6,125	Load A	3,062
Load B	6,325	Load B	3,162
Load C	6,700	Load C	3,350
Load D	7,080	Load D	3,540

[1] These are the customary designations of the tires listed, as used by the Tire and Rim Association and by the industry in general. High pressure tires are designated by over-all height in inches and section height in inches and fractions thereof. Balloon tires are designated by section height in inches and decimals thereof and rim diameter in inches. In the case of the 9.00 by 20 inch balloons, where two rim widths were used, these are given.

Natural road surface roughness conditions were used in making tests to determine the actual magnitudes of impact forces developed under normal operating conditions. These tests were made in the vicinity of Washington, D. C., at locations where the desired speeds could be attained with safety. The natural roughness conditions which were found at places which were practicable for making tests were of less severity than the artificial obstructions described in a preceding paragraph. Profiles of the natural obstructions are given later in this report in presenting the data obtained with them.

Vertical impact reactions occurring between a moving wheel and the pavement surface may be resolved into two components. One is the dynamic force due to the vertical acceleration or deceleration imparted to the unsprung weight at the wheel and the other is the static weight on that wheel.

In the impact investigations of the bureau the dynamic force has been obtained by multiplying the value of the unsprung mass by the acceleration developed at the instant of impact. The mass values were obtained by weighing and the acceleration by measurement with an accelerometer. By adding to this dynamic force the static load on the wheel, the total vertical

impact reaction was obtained and may be expressed in pound units or as a percentage of the static wheel load.[2]

MULTIPLE ELEMENT CONTACT ACCELEROMETER USED TO MEASURE IMPACT FORCES

The multiple element contact accelerometer used in these tests was designed and built by the Bureau of Public Roads as a modification of a single element instrument of the same type described in the July, 1930, issue of Public Roads. It differs from the original instrument in that it comprises 10 sensitive elements equipped with styli for direct recording on a moving ribbon of paraffin-coated colored paper. The hardened steel styli have ball points and the elements are arranged in offset relation so that the record made by one element does not interfere with that made by another. The movement of each sensitive element measured at its center of gravity was restricted to 0.001 inch between the recording and non-recording positions of its stylus. The accelerometer was mounted on the rear axle of the bus, the frame supporting it being clamped securely to the top of the right hand spring bolster by the bolts which held the spring to the axle. It was thus in a position to respond to any vertical accelerations imparted to the axle by the wheel. A general view of the instrument is given in Figure 4.

The sensitive elements were calibrated statically and dynamically according to methods described in the earlier publication.[3] As the dynamic calibration involved the use of a telephone circuit to detect movement of the sensitive elements, additional tests were made to prove that the autographic recording on paraffin-coated paper is as accurate and reliable as that involving the telephone circuit. A further check on the calibration was obtained by comparing the experimental values with the design or theoretical values. The characteristics of the 10 elements used are given in Table 5.

TABLE 5.—*Characteristics of accelerometer elements*

Element No.	Thickness of spring	Calibration rate	Free period
	Inches	*Feet per sec.² per 0.001 inch deflection* [1]	*Seconds*
1	0.0381	2.79	0.0071
2	.0402	3.26	.0067
3	.0410	3.52	.0066
4	.0425	3.85	.0063
5	.0437	4.26	.0062
6	.0451	4.53	.0060
7	.0458	4.72	.0059
8	.0470	5.00	.0057
9	.0480	5.48	.0056
10	.0491	5.78	.0055

[1] Measured at the micrometer.

Accelerometers of the contact type are so designed that the action of any one sensitive element will only indicate whether an acceleration greater or less than a certain magnitude was attained during impact. That critical acceleration magnitude is determined by the resistance to be overcome (which, in the accelerometer used is caused by the deflection of a cantilever spring) and the mass of the sensitive element acted upon by the impressed acceleration. By placing a battery of such elements in a single instrument unit and setting each element to respond to a different magnitude of acceleration, an impressed acceleration may be determined as lying between certain limits which are indicated by the critical values of the elements which do or do not respond. In the tests herein described, the critical accelerations to which the various elements were set to respond varied in increments of 10 feet per second per second, and the range in acceleration thus covered by the instrument unit at any one time was, therefore, 90 feet per second per second.

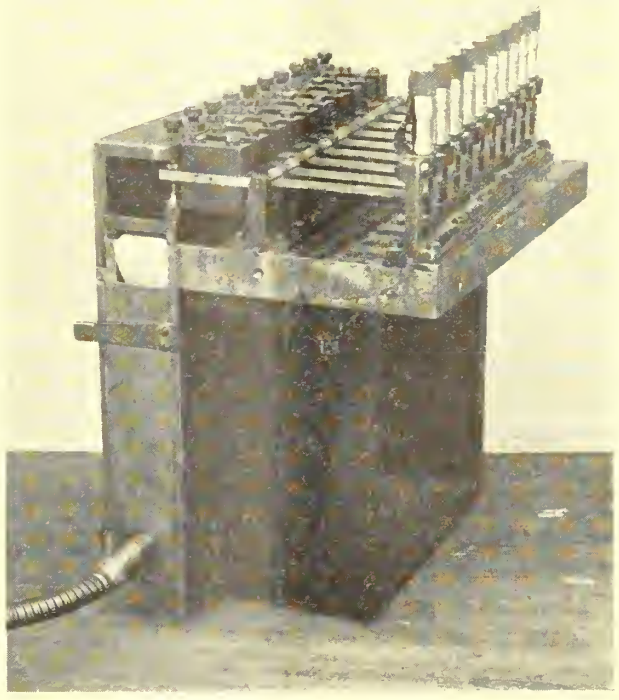

FIGURE 4.—THE AUTOGRAPHIC MULTIPLE-ELEMENT CONTACT ACCELEROMETER

If the acceleration attained in a given impact test did not fall within the range covered, as indicated by the fact that all the elements responded or failed to respond, that test was repeated with the elements set to include a higher or lower group of critical accelerations as the case might be. If some of the elements responded, Nos. 1 to 3 by way of example, and the remainder, Nos. 4 to 10, did not respond, then the acceleration for that test was determined as lying between the critical values for which elements Nos. 3 and 4 had been set. It frequently happened that when the instrument was set to record the acceleration produced by one of the two artificial obstructions it would also respond to that produced by the other. Thus one acceleration might be determined by elements 3 and 4 and the other by elements 8 and 9. In this manner an opportunity was afforded to check the result thus obtained by chance with that obtained when the instrument was set to record the acceleration produced by the other obstruction. The checks thus obtained were always very close.

[2] To be strictly accurate, some allowance should be made for the variation in pressure transmitted through the vehicle spring as it flexes during the impact phenomenon. This had been determined in earlier tests (see Public Roads, Vol. 7, No. 4, June, 1926) and found to exert comparatively little influence. Since comparative tests would all be slightly modified by approximately equal amounts, the determination, by measurement, of variations in spring pressure was not considered to be justified in this series of tests.

[3] See Public Roads, July, 1930.

DISCUSSION OF THE DATA

Typical curves showing results obtained with the artifical obstructions are given in Figure 5. It is noted that where drop conditions exist a maximum reaction is reached at a comparatively low speed and that the reaction generally decreases as the speed is increased above this critical value. The interaction between the sprung and unsprung weights of the vehicle while the wheel is in the air and falling is probably the reason for this condition.

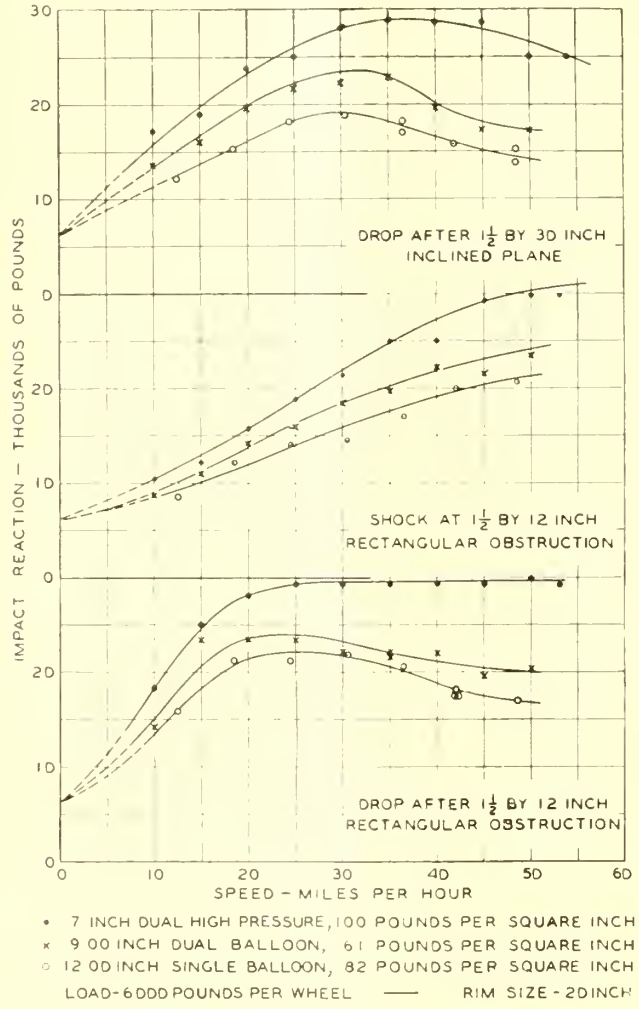
IMPACT REACTION — THOUSANDS OF POUNDS
SPEED - MILES PER HOUR
DROP AFTER 1½ BY 30 INCH INCLINED PLANE
SHOCK AT 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
DROP AFTER 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
• 7 INCH DUAL HIGH PRESSURE, 100 POUNDS PER SQUARE INCH
× 9 00 INCH DUAL BALLOON, 61 POUNDS PER SQUARE INCH
○ 12 00 INCH SINGLE BALLOON, 82 POUNDS PER SQUARE INCH
LOAD - 6000 POUNDS PER WHEEL —— RIM SIZE - 20 INCH

FIGURE 5.—IMPACT REACTIONS FOR TYPICAL TIRE EQUIPMENTS PASSING OVER THE ARTIFICIAL OBSTRUCTIONS

The reactions under shock conditions are found to increase in approximate proportion with vehicle speed. When the wheel encounters an obstruction under shock conditions, there is a tendency for it to be raised above the surface of the road. As the vehicle speed increases this elevation must necessarily take place in a shorter time interval, resulting in greater vertical accelerations with corresponding increases in impact reaction. The maximum reaction obviously occurs when a given obstruction causes a maximum deformation in the tire. It is reasonable to suppose that this maximum deformation occurs when the tire completely envelops the obstruction without appreciably increasing the elevation of the axle. An approach to this maximum condition is noted in the tendency for the curves to flatten at higher speeds, indicated in the central section of Figure 5.[4]

The two lower sections of Figure 5 are for shock at and drop after a given obstruction. It is noted that the values of the drop reactions obtained at relatively low speeds are not equaled or exceeded by the corresponding shock reactions except at relatively high vehicle speeds. For the particular vehicle and other test conditions given these speeds are from 20 to 30 miles per hour for the maximum drop reaction and 50 to 60 miles per hour for shock reactions of equal magnitude.

BALLOON TIRE EQUIPMENT REDUCES IMPACT REACTIONS

One of the questions most frequently asked is, "What is the relative influence of the type of tire equipment on impact reaction?" In Figure 5, data are given for three types of impact reaction, comparing the forces produced on the pavement by the same wheel when equipped with dual high pressure pneumatic, dual balloon, and single balloon tires of equal capacity and carrying the same load. There is no question as to the additional protection to the road which results from replacing high pressure pneumatic tires with balloon tires. The balloon tire operates at a lower inflation pressure and has a greater section height than the high pressure tire of equal capacity, both of which factors enhance its cushioning properties. Since protection to the road is also protection to the vehicle this comparison has a major economic significance. The road strikes the vehicle wheel just as severely as the wheel strikes the road.

SINGLE BALLOON TIRE PRODUCES LOWER IMPACT FORCES THAN CORRESPONDING DUAL MOUNTING, BUT GREATER CONCENTRATION OF PRESSURE RESULTS

The relation between dual and single tire equipments of the same type and of equal load carrying capacity is also brought out in Figure 5. Although the 12.00-inch balloon requires an inflation pressure of 82 pounds per square inch as compared with 61 pounds per square inch required for dual 9.00-inch tires to carry the same load, the greater section height of the single tire results in lower impact forces than those caused by the smaller-sectioned dual tires. This observation supports a similar conclusion reached as a result of the earlier impact tests with motor trucks and reported in the June, 1926, issue of Public Roads.

In connection with this discussion of dual versus single tire equipments, it is important to consider the intensity of contact pressures developed by the two types. As shown by Table 6, which gives results obtained from static tests on the three equipments involved in Figure 5, the single balloon tire develops considerably higher average contact pressures than do the dual balloon tires. The data given in Table 6 are based on measurements of contact area and contact width as given by the three tire types under static loads.

The static load-deflection characteristics of the three tire equipments discussed above are given in Figure 6,

[4] In the article entitled "Calibrations of Accelerometers for Use in Motor Truck Impact Tests," Public Roads, Vol. 11, No. 5, July, 1930, the nature of the shock reaction is discussed from a theoretical standpoint. It is believed that the theory developed, although the assumptions on which it is based are admittedly approximate, indicates, qualitatively, the influence of the several variables involved in the shock impact. On the basis of this theory the limiting value of the shock reaction is given by the equation, $F = L + K\sqrt{h}$, where L is the wheel load, K is a factor expressing the resistance of the tire to the vertical deformation caused by the obstruction, and h is the height of the obstruction.

TABLE 6.—*Comparison of contact pressures developed by dual high pressure, dual balloon, and single balloon tires*

Tire size	Inflation pressure	Load per wheel	Unit pressure, based on gross contact area	Average load per unit contact width
Inches	*Lbs. per sq. in.*	*Pounds*	*Lbs. per sq. in.*	*Lbs. per in.*
Dual 34 by 7	100	6,000	86	581
		12,000	103	1,012
		18,000	118	1,488
Dual 9.00 by 20	61	6,000	59	485
		12,000	75	936
		18,000	85	1,353
Single 12.00 by 20	82	6,000	76	761
		12,000	90	1,264
		18,000	101	1,811

for purposes of comparison. It is at once apparent that, for comparable operating conditions, the relation between the static load-deflection characteristics of the tires is a fair index of the relative impact reactions produced by them.

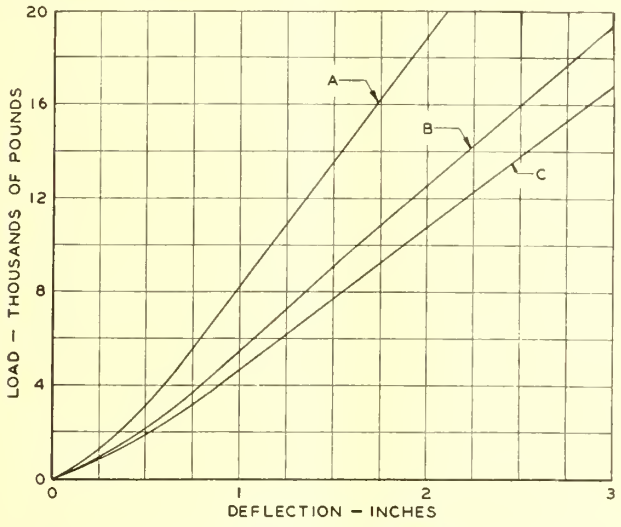
LOAD — THOUSANDS OF POUNDS
DEFLECTION — INCHES
A
B
C
A- 7 INCH DUAL HIGH PRESSURE, 100 POUNDS PER SQUARE INCH
B- 9.00 INCH DUAL BALLOON, 61 POUNDS PER SQUARE INCH
C-12.00 INCH SINGLE BALLOON, 82 POUNDS PER SQUARE INCH

FIGURE 6.—STATIC LOAD-DEFLECTION CURVES SHOWING INFLUENCE OF TIRE TYPE

VARIATION OF IMPACT FORCE WITH INFLATION PRESSURE ANALYZED

Closely related to the influence of tire type is that of inflation pressure. It was noted in the discussion of Figure 5 that maximum reactions were obtained at relatively low speeds for the drop impact conditions. In Figure 7 the magnitudes of these maximum reactions have been plotted against corresponding variations in inflation pressures and curves have been drawn for the two drop impact conditions. The wheel load was 6,000 pounds in each case and the speed was that at which the maximum drop reaction occurred. The curves have been broken between points representing the two tire equipments represented by these data, the higher inflation pressures having been applied to dual 34 by 7 inch high-pressure tires and the lower pressures to dual 9.00 by 20 inch balloon tires, the two tire equipments being replacement sizes for each other. It is at once noted that the variation in maximum impact reaction is approximately proportional to the variation in inflation pressure.

For purposes of comparison, the static reactions corresponding to a deflection of 1½ inches have also been plotted against the corresponding inflation pressures in Figure 7. The analogy between relations brought out by static and impact tests is again apparent.

The influence of inflation pressure is brought out in a different way in Figure 8. In this figure, which involves the three types of impact reaction, data are plotted for all tire equipments which were tested under the load and inflation standards set forth in Tables 2 and 3. The curves which have been drawn may be regarded as typical for high pressure and balloon tires under the respective roughness conditions involved. In this

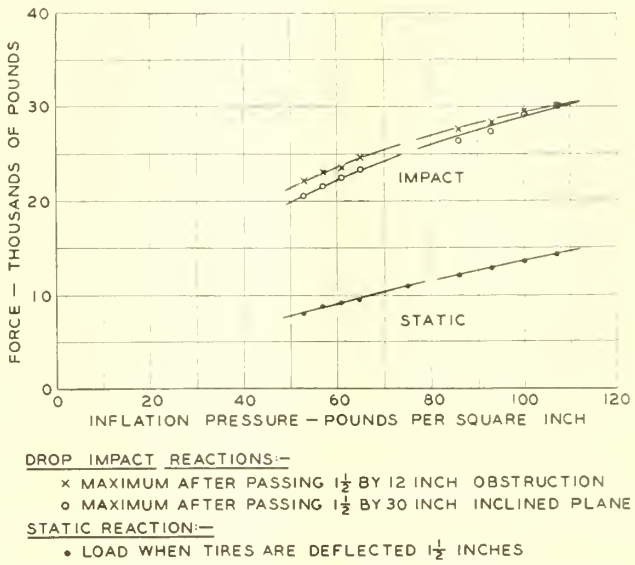
FORCE — THOUSANDS OF POUNDS
INFLATION PRESSURE — POUNDS PER SQUARE INCH
IMPACT
STATIC
DROP IMPACT REACTIONS:—
× MAXIMUM AFTER PASSING 1½ BY 12 INCH OBSTRUCTION
o MAXIMUM AFTER PASSING 1½ BY 30 INCH INCLINED PLANE
STATIC REACTION:—
• LOAD WHEN TIRES ARE DEFLECTED 1½ INCHES

FIGURE 7.—INFLUENCE OF INFLATION PRESSURE ON TIRE REACTIONS

figure the mean ratio of the dynamic increments (i. e., ordinates above the 100 per cent static load line) for balloon tires to the corresponding increments for high pressure tires is about 0.62; while the mean ratio of the total impact reactions is about 0.72. The mean ratio between the inflation pressures recommend in Table 2 to carry a given load on balloon tires and those recommended in Table 3 for high-pressure tires of equal base (rim) diameter at the same load was found to be about 0.63. It may, therefore, be concluded that, for a given roughness condition and at a given speed, balloon and high-pressure tire equipments will develop impact reactions, in excess of static load, approximately in the proportion of the inflation pressures used, where the load carried is the same in each case and recommended inflation pressures are used. According to current standards this proportion is of the order of two to three.

EFFECT OF LOAD STUDIED

In Figures 9 to 13, data are given which show the influence of load on impact reaction. The three sections in each figure refer to the three types of reaction caused by the artificial obstructions. In Figures 9 and 10, impact forces have been expressed in pound units and in Figures 11 and 12 they have been plotted as a percentage of the particular wheel load carried. In these tests, the inflation pressure in a given tire was maintained constant as the load was varied, 100 pounds per square inch being applied to the dual 34 by 7 inch high-

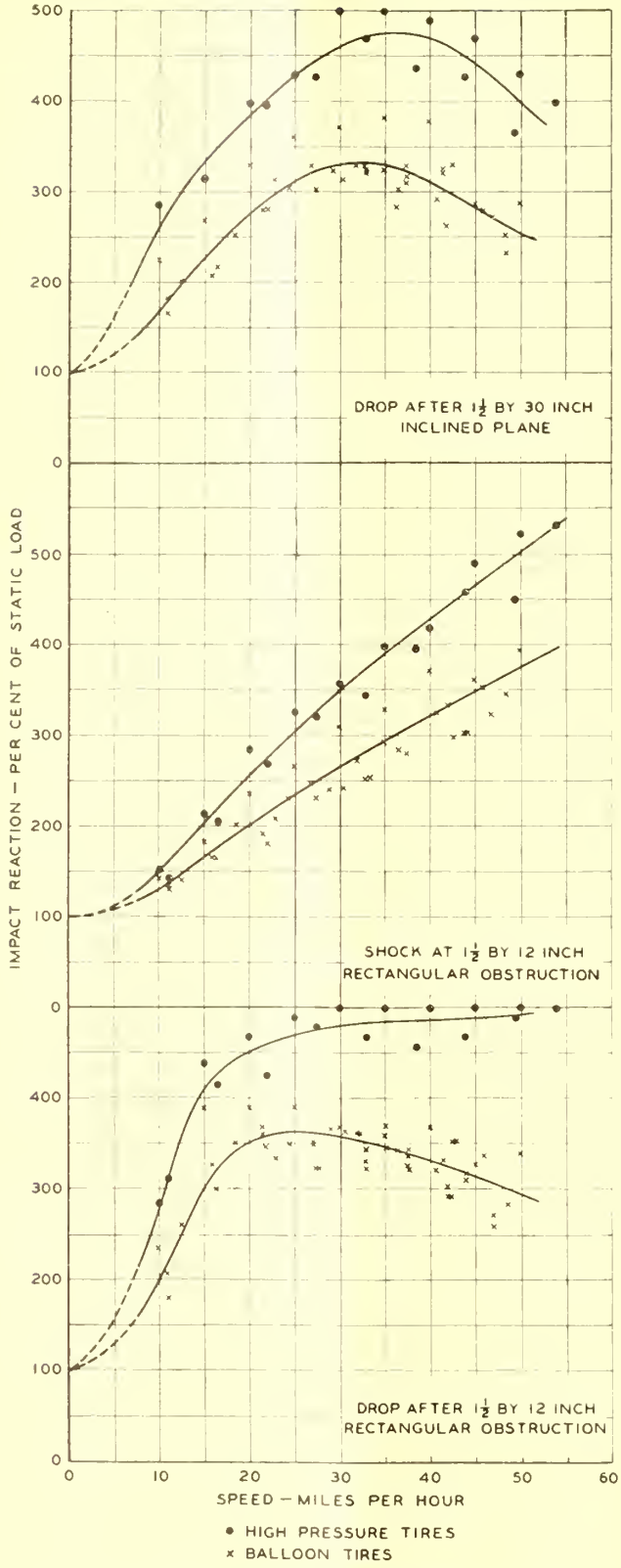
IMPACT REACTION — PER CENT OF STATIC LOAD
DROP AFTER 1½ BY 30 INCH INCLINED PLANE
SHOCK AT 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
DROP AFTER 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
SPEED — MILES PER HOUR
• HIGH PRESSURE TIRES
× BALLOON TIRES

FIGURE 8.—BALLOON VERSUS HIGH-PRESSURE TIRES: IMPACT REACTIONS PRODUCED BY ARTIFICIAL OBSTRUCTIONS FOR ALL TIRE EQUIPMENTS TESTED AT RECOMMENDED LOADS AND INFLATION PRESSURES

pressure pneumatics and 65 pounds per square inch to the dual 9.00 by 20 inch balloons. In Figure 13, com-

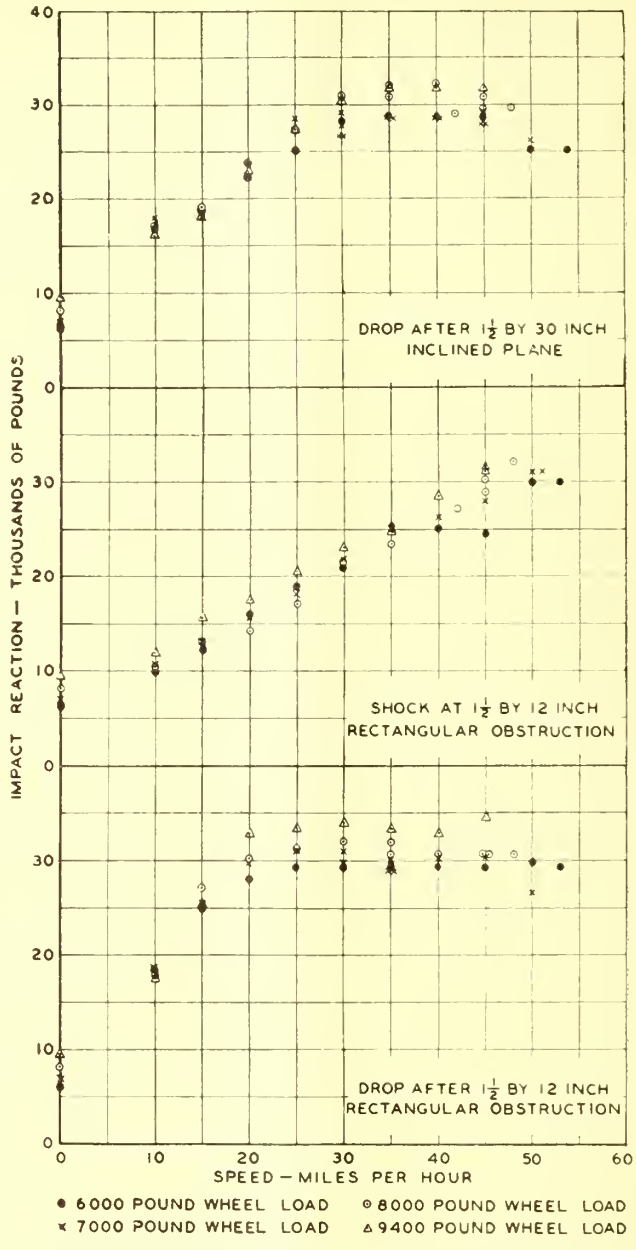
IMPACT REACTION — THOUSANDS OF POUNDS
DROP AFTER 1½ BY 30 INCH INCLINED PLANE
SHOCK AT 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
DROP AFTER 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
SPEED — MILES PER HOUR
• 6000 POUND WHEEL LOAD
⊙ 8000 POUND WHEEL LOAD
× 7000 POUND WHEEL LOAD
△ 9400 POUND WHEEL LOAD

FIGURE 9.—INFLUENCE OF LOAD ON THE IMPACT REACTIONS PRODUCED BY DUAL 34 BY 7 INCH HIGH-PRESSURE TIRES AT AN INFLATION PRESSURE OF 100 POUNDS PER SQUARE INCH. PLOTTED POINTS DENOTE VALUES OF THE IMPACT REACTIONS IN THOUSANDS OF POUNDS

parisons are made between tire equipments seriously overloaded and larger equipment designed to carry such loads.

For the test conditions covered by Figures 9 and 10, it is noted that load has comparatively little influence on the magnitude of the impact reaction between the wheel and the pavement. The heavier loads have some slight tendency toward increased reactions, but this increase scarcely exceeds the difference in static wheel load. It is indicated that, within the load range of these tests, the dynamic increment, i. e., the increase in force above that due to static load, is approximately constant for a given tire and inflation condition and for stated speed and roughness conditions.

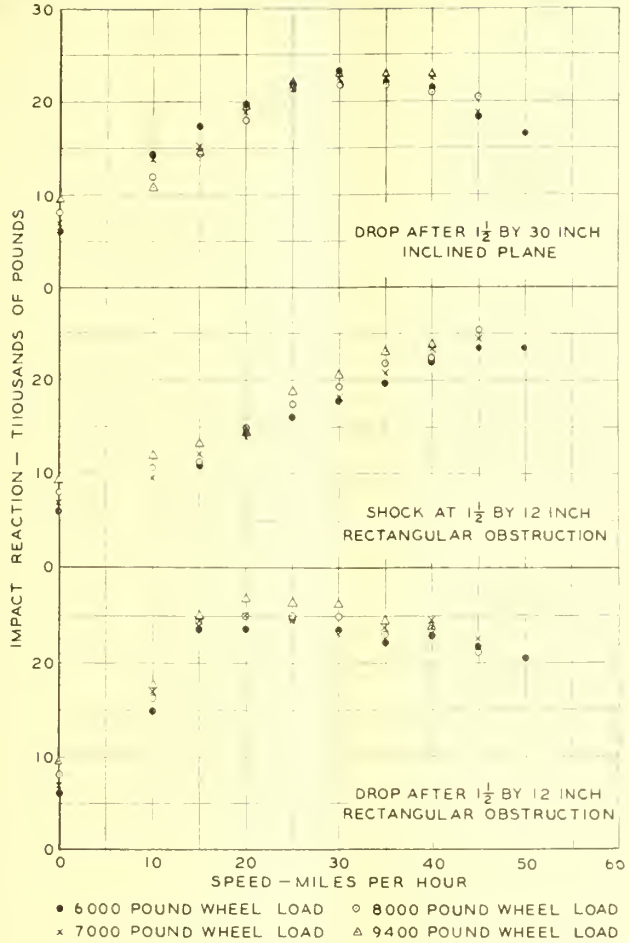
IMPACT REACTION — THOUSANDS OF POUNDS
DROP AFTER 1½ BY 30 INCH INCLINED PLANE
SHOCK AT 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
DROP AFTER 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
SPEED — MILES PER HOUR
• 6000 POUND WHEEL LOAD ○ 8000 POUND WHEEL LOAD
× 7000 POUND WHEEL LOAD △ 9400 POUND WHEEL LOAD

FIGURE 10.—INFLUENCE OF LOAD ON THE IMPACT REACTIONS PRODUCED BY DUAL 9.00 BY 20 INCH BALLOON TIRES AT AN INFLATION PRESSURE OF 65 POUNDS PER SQUARE INCH. PLOTTED POINTS DENOTE VALUES OF THE IMPACT REACTION IN THOUSANDS OF POUNDS

In Figures 11 and 12, the impact reactions corresponding to those in Figures 9 and 10 have been plotted on the basis of their percentage of the static wheel loads used. It is at once noted that on this basis and for the conditions covered by these tests there is a marked decrease as the wheel load is increased. These indications which are so clearly brought out in Figures 9 to 12 are in accordance with those previously observed in the report on motor truck impact tests in the June, 1926, issue of Public Roads.

In Figure 13 the reactions obtained with dual 10.50 by 20 inch balloons which are designed to carry a wheel load of 9,400 pounds at the recommended inflation pressure of 75 pounds per square inch are compared with the reactions obtained with dual 34 by 7 inch high-pressure tires at 100 pounds per square inch and dual 9.00 by 20 inch balloon tires at 65 pounds per square inch, each of the two latter being overloaded approximately 50 per cent to carry the 9,400-pound wheel load. In the earlier experiments [5] the conclusion was reached that, for a given type of tire equipment, a smaller size operating overloaded causes lower reactions than a larger size operating at that same load. Referring to Figure 13, we confirm this conclusion as applying to the balloon type, lower reactions being caused by the overloaded 9.00 by 20 inch tires than are by the properly loaded 10.50 by 20 inch tires, the same wheel load being carried in each case.

[5] See Public Roads, June, 1926.

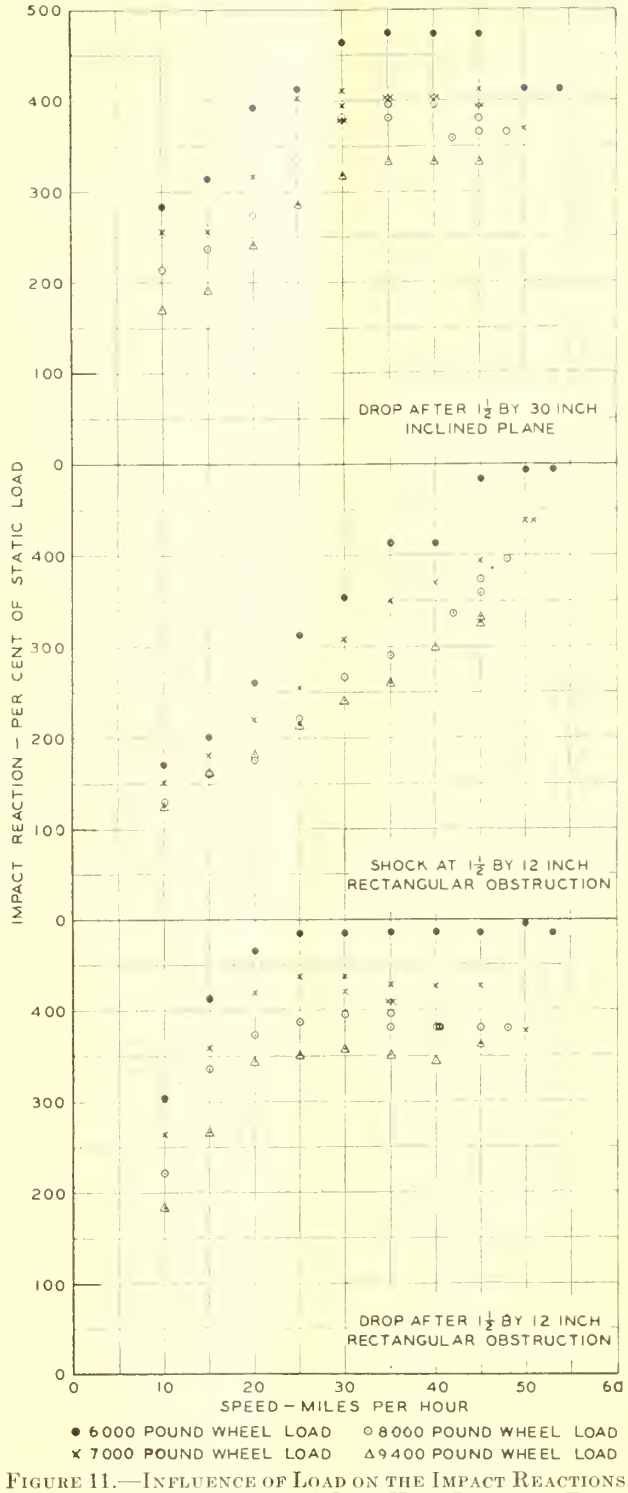
IMPACT REACTION — PER CENT OF STATIC LOAD
DROP AFTER 1½ BY 30 INCH INCLINED PLANE
SHOCK AT 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
DROP AFTER 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
SPEED — MILES PER HOUR
• 6000 POUND WHEEL LOAD ○ 8000 POUND WHEEL LOAD
× 7000 POUND WHEEL LOAD △ 9400 POUND WHEEL LOAD

FIGURE 11.—INFLUENCE OF LOAD ON THE IMPACT REACTIONS PRODUCED BY DUAL 34 BY 7 INCH HIGH-PRESSURE TIRES AT AN INFLATION PRESSURE OF 100 POUNDS PER SQUARE INCH. PLOTTED POINTS DENOTE VALUES OF THE IMPACT REACTION IN PERCENTAGE OF STATIC LOAD

In a preceding paragraph it was brought out that impact reactions are approximately in proportion to the inflation pressure used in a pneumatic tire. It should therefore be expected that balloon tires would cause lower reactions than comparable high-pressure tires having the same load-carrying capacity, since the balloon tires operate at considerably lower inflation pressures. This is borne out in Figure 13, the 9.00 by 20 inch tires causing lower reactions than the 34 by 7 inch tires under the same load conditions.

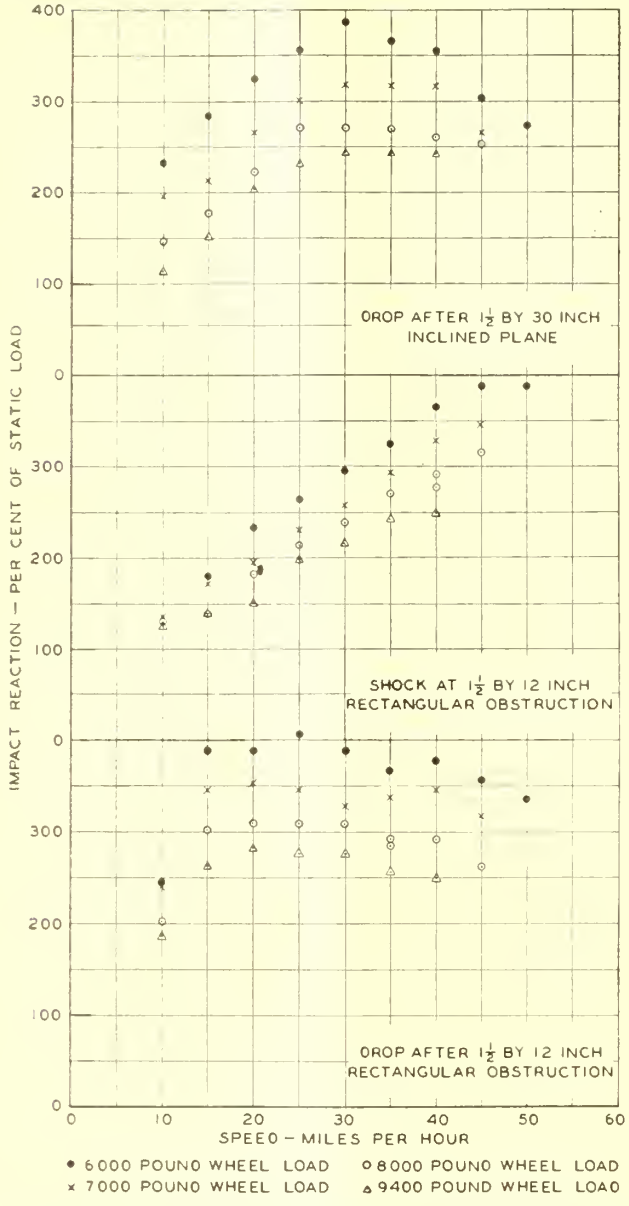
IMPACT REACTION — PER CENT OF STATIC LOAD
DROP AFTER 1½ BY 30 INCH INCLINED PLANE
SHOCK AT 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
DROP AFTER 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
SPEED — MILES PER HOUR
• 6000 POUND WHEEL LOAD
o 8000 POUND WHEEL LOAD
× 7000 POUND WHEEL LOAD
▵ 9400 POUND WHEEL LOAD

FIGURE 12.—INFLUENCE OF LOAD ON THE IMPACT REACTIONS PRODUCED BY DUAL 9.00 BY 20 INCH BALLOON TIRES AT AN INFLATION PRESSURE OF 65 POUNDS PER SQUARE INCH. PLOTTED POINTS DENOTE VALUES OF THE IMPACT REACTION IN PERCENTAGE OF STATIC LOAD

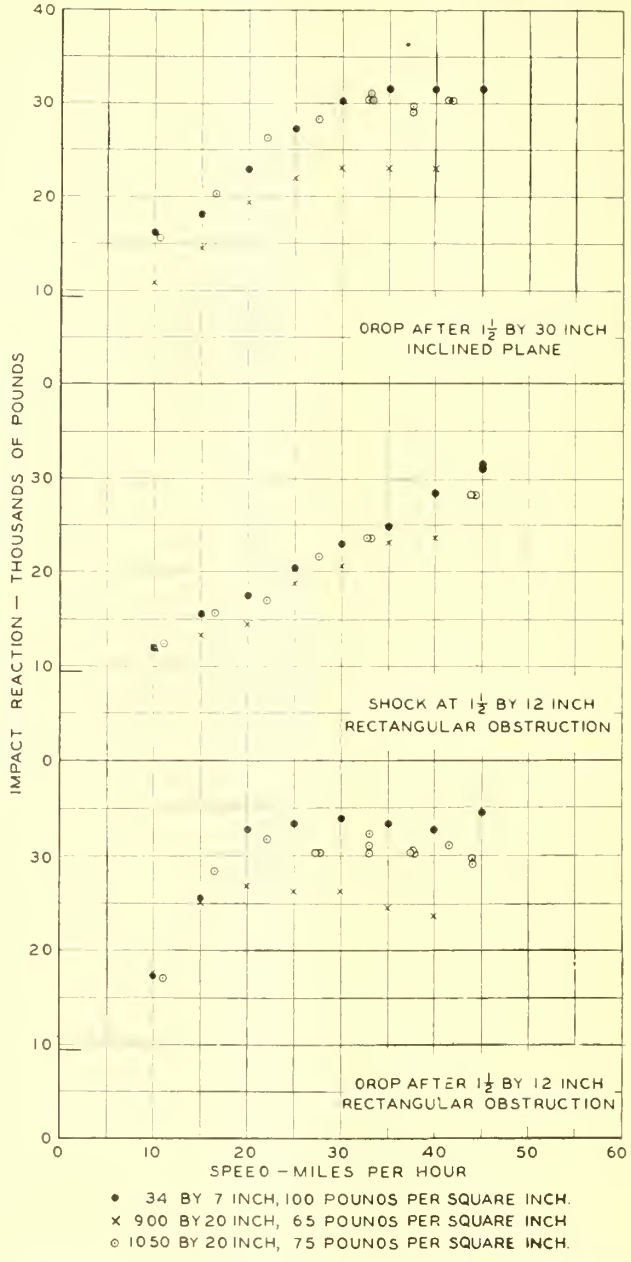
IMPACT REACTION — THOUSANDS OF POUNDS
DROP AFTER 1½ BY 30 INCH INCLINED PLANE
SHOCK AT 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
DROP AFTER 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
SPEED — MILES PER HOUR
• 34 BY 7 INCH, 100 POUNDS PER SQUARE INCH.
× 900 BY 20 INCH, 65 POUNDS PER SQUARE INCH
o 1050 BY 20 INCH, 75 POUNDS PER SQUARE INCH.

FIGURE 13.—COMPARISON OF THE IMPACT REACTIONS PRODUCED BY WHEELS LOADED TO 9,400 POUNDS AND EQUIPPED RESPECTIVELY, WITH OVERLOADED 34 BY 7 INCH HIGH-PRESSURE TIRES, OVERLOADED 9.00 BY 20 INCH BALLOON TIRES, AND NORMALLY LOADED 10.50 BY 20 INCH BALLOON TIRES, ALL DUAL MOUNTINGS

It has just been shown that the 9.00 by 20 inch overloaded balloon tires cause lower reactions than either the 10.50 by 20 inch normally loaded balloon tires or the 34 by 7 inch overloaded high-pressure tires, the same wheel loads being carried by each. It is interesting to note in Figure 13 that the reactions caused by the 34 by 7 inch high-pressure tires and those caused by the 10.50 by 20 inch balloon tires were approximately the same. It appears, then, that for the tire equipments and other test conditions involved the overloaded 9.00 by 20 inch balloon tires caused lower reactions than the properly loaded 10.50 by 20 inch balloon tires in about the same degree that the overloaded balloon type caused lower reactions than the overloaded high-pressure type.

The impression should not be given from the data in Figures 9 to 13 that overloading is a practice to be cultivated. While it is true that lower magnitudes of impact reaction are obtained with overloaded tires than when tires of adequate capacity are used, other factors should be considered. One factor which is of particular importance to highway engineers is the increased intensity of road contact pressure which results from overloading. Table 7, based on static tests, gives average loads per unit of gross area of contact and per unit of width of contact for the three tire equipments under consideration. The increased concentration due to overloading is apparent.

Another factor to be considered before overloading is permitted is the decrease in tire mileage under operating conditions involving heavier loads than those for which tires are designed. According to statements of tire manufacturers, the practice of loading tires to 150 per cent of their recommended capacities results

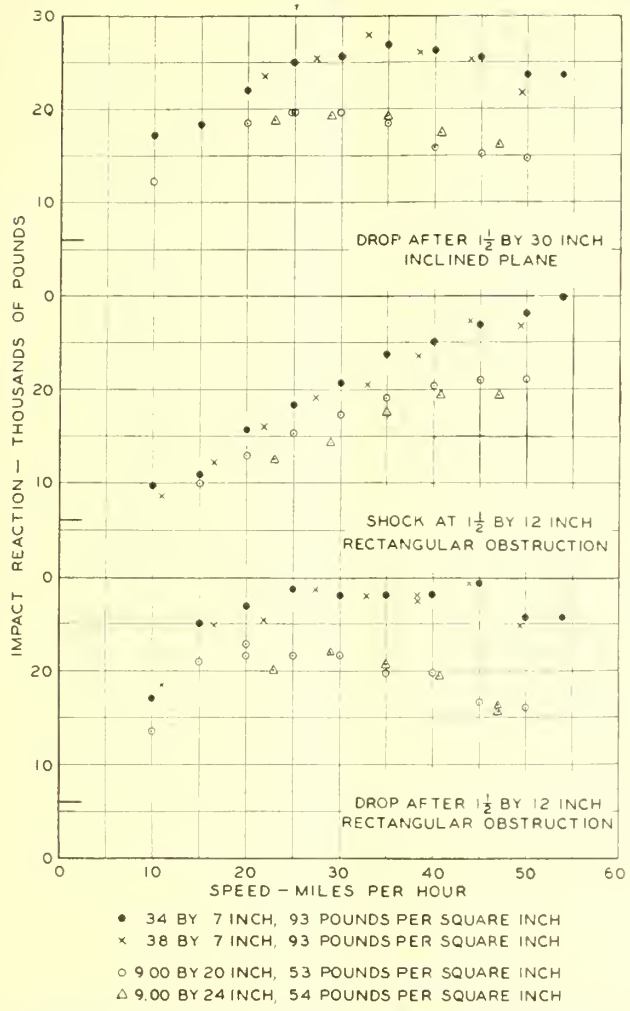
IMPACT REACTION — THOUSANDS OF POUNDS
SPEED — MILES PER HOUR
DROP AFTER 1½ BY 30 INCH INCLINED PLANE
SHOCK AT 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
DROP AFTER 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
• 34 BY 7 INCH, 93 POUNDS PER SQUARE INCH
× 38 BY 7 INCH, 93 POUNDS PER SQUARE INCH
○ 9.00 BY 20 INCH, 53 POUNDS PER SQUARE INCH
△ 9.00 BY 24 INCH, 54 POUNDS PER SQUARE INCH

FIGURE 14.—INFLUENCE OF RIM DIAMETER ON THE IMPACT REACTIONS PRODUCED BY HIGH PRESSURE AND BALLOON TIRES, THE TWO TIRES OF EACH TYPE BEING INFLATED TO THE SAME PRESSURE. WHEEL LOAD, 6,000 POUNDS

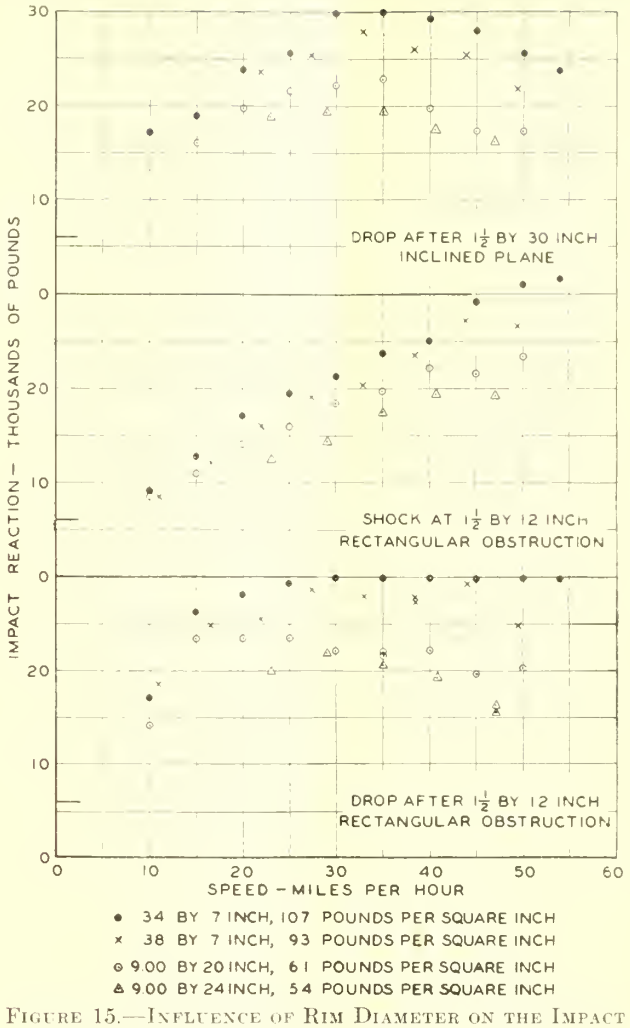
IMPACT REACTION — THOUSANDS OF POUNDS
SPEED — MILES PER HOUR
DROP AFTER 1½ BY 30 INCH INCLINED PLANE
SHOCK AT 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
DROP AFTER 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
• 34 BY 7 INCH, 107 POUNDS PER SQUARE INCH
× 38 BY 7 INCH, 93 POUNDS PER SQUARE INCH
⊙ 9.00 BY 20 INCH, 61 POUNDS PER SQUARE INCH
▲ 9.00 BY 24 INCH, 54 POUNDS PER SQUARE INCH

FIGURE 15.—INFLUENCE OF RIM DIAMETER ON THE IMPACT REACTIONS PRODUCED BY HIGH-PRESSURE AND BALLOON TIRES, EACH TIRE BEING INFLATED TO ITS RECOMMENDED PRESSURE. WHEEL LOAD, 6,000 POUNDS

in a decrease of approximately 50 per cent in the mileage they would be expected to give in normal service.

EFFECT OF RIM DIAMETER NEGLIGIBLE IF INFLATION PRESSURES ARE EQUAL

The influence of rim diameter on impact reactions produced by tires of a given sectional size is given in Figures 14 and 15. Most, of not all, of the high-pressure and balloon tires in truck and bus service in this country to-day are for appliance on rims having diameters of 20, 22, or 24 inches. This results in a variation in rolling radius of approximately two inches for a given sectional size of tire. In Figure 14 are given the reactions obtained with 7-inch high-pressure and 9.00-inch balloon sections applied to 20 and 24 inch diameter rims, the inflation pressure being 93 pounds per square inch for the 7-inch sections and 53 to 54 pounds per square inch for the 9.00-inch sections. It may be stated that, within the scope of these tests, there is no important difference in impact reaction between tires of a given cross-section but differing by 4 inches in diameter, when the inflation pressures are equal.

TABLE 7.—*Comparison of contact pressures developed by 34 by 7 inch high pressure, 9.00 by 20 inch balloon, and 10.50 by 20 inch balloon tires, under equal loads*

Tire size	Inflation pressure	Load per tire	Unit pressure, based on gross contact area	Average load per unit contact width
Inches	*Lbs. per sq. in.*	*Pounds*	*Lbs. per sq. in.*	*Lbs. per in.*
34 by 7	100	[1] 3,000	86	581
		4,700	[2] 96	[2] 825
		6,000	103	1,012
		9,000	118	1,488
		9,400	[3] 120	[3] 1,550
9.00 by 20	65	[1] 3,000	62	498
		4,700	[2] 72	[2] 750
		6,000	78	933
		9,000	89	1,360
		9,400	[3] 90	[3] 1,420
10.50 by 20	75	[1] 4,700	78	711
		9,400	88	1,124
		14,100	100	1,621

[1] Approximate capacity load. [2] Interpolated. [3] Extrapolated

Referring back to Tables 2 and 3, we note that the recommended inflation pressure for tires of a given section to carry a given load is less for those mounted on larger diameter rims than for those mounted on smaller diameters. In Figure 15, another comparison is made of the reactions caused by 7-inch high-pressure and 9.00-inch balloon tires on 20 and 24-inch rims. The data here presented were obtained from tests in which each tire carried the recommended pressure for the given load. In this case the reactions involving the larger rim diameter are noticeably less than those

involving the smaller, and taken in conjunction with the study of Figure 14, this decrease may be largely attributed to the lower inflation pressures used with the larger diameter tires.

The static load-deflection curves given in Figure 16 support the impact reaction data given in Figures 14 and 15. Here again it is noted that where the pressure is the same there is but little difference between the behavior of tires of a given section but of different diameters.

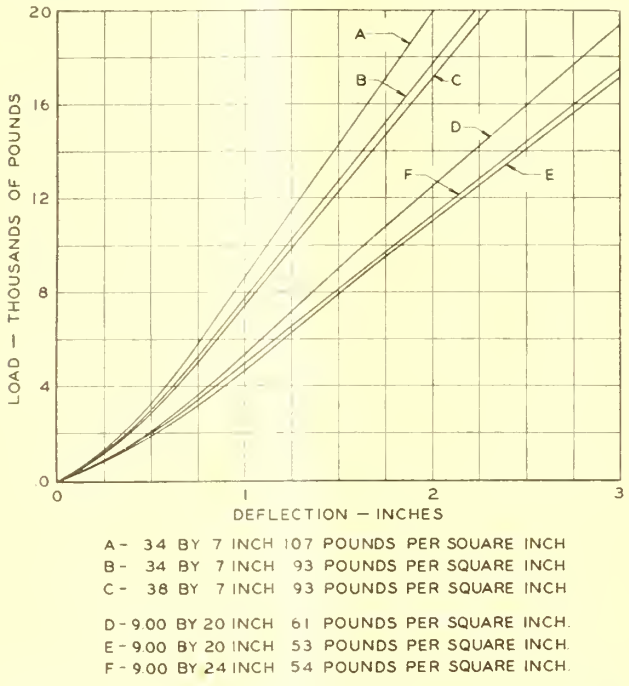
LOAD — THOUSANDS OF POUNDS
DEFLECTION — INCHES
A
B
C
D
E
F
A - 34 BY 7 INCH 107 POUNDS PER SQUARE INCH
B - 34 BY 7 INCH 93 POUNDS PER SQUARE INCH
C - 38 BY 7 INCH 93 POUNDS PER SQUARE INCH
D - 9.00 BY 20 INCH 61 POUNDS PER SQUARE INCH
E - 9.00 BY 20 INCH 53 POUNDS PER SQUARE INCH
F - 9.00 BY 24 INCH 54 POUNDS PER SQUARE INCH

FIGURE 16.—STATIC LOAD-DEFLECTION CURVES SHOWING THE INFLUENCE OF RIM DIAMETER ON HIGH-PRESSURE AND BALLOON TIRES

VARIATION IN RIM WIDTH HAS NO APPRECIABLE INFLUENCE

It frequently happens that a given tire may properly be applied to rims of different widths. For instance, if the original tire equipment on a vehicle were 34 by 7, 7.50 by 20, or 8.25 by 20 inch, the original rim equipment would be 20 by 7 inch. Should it be desirable to use 9.00 by 20 inch tires on that vehicle, accepted practice would permit the application of such tires to the 20 by 7 inch rims provided other factors such as clearance and gear ratio did not interfere. However, if the vehicle had been originally equipped with 36 by 8 or 9.00 by 20 inch tires the rim equipment would have been 20 by 8 inch. The inflation pressure recommended for the 9.00 by 20 inch tires would be the same whether mounted on either rim.

The impact reactions obtained with dual 9.00 by 20 inch tires mounted on 7 and 8 inch rims are given in Figure 17 and the corresponding static load-deflection curves are in Figure 18. It is at once apparent that this variation of one inch in rim width causes no appreciable change in the action of the tires.

COMPARISONS MADE WITH DATA FROM EARLIER TRUCK IMPACT TESTS

At this point it would be well to compare the impact reactions obtained in these tests with the bus chassis at relatively high speeds with those obtained in the earlier tests with trucks at relatively low speeds. In both series of tests an inclined plane artificial obstruction 1½ inches in height and 30 inches in length was involved. For this roughness condition and for comparable tires and loads, results from the bus and truck tests have been plotted in Figures 19, 20, and 21. The data concerning trucks in Figures 19 and 20 were obtained from tests made with a 3-ton chassis with 6,000 and 7,000 pound wheel loads, respectively; while in Figure 21 they concern 8,400-pound wheel loads on a 5-ton chassis. In the bus tests the same chassis was used for all loads.

The impact reactions for the truck tests were computed by the method allowing for variations in truck spring pressure (see Public Roads, June, 1926), while in the bus tests a constant pressure equal to the pressure at static load was used in computation. Although

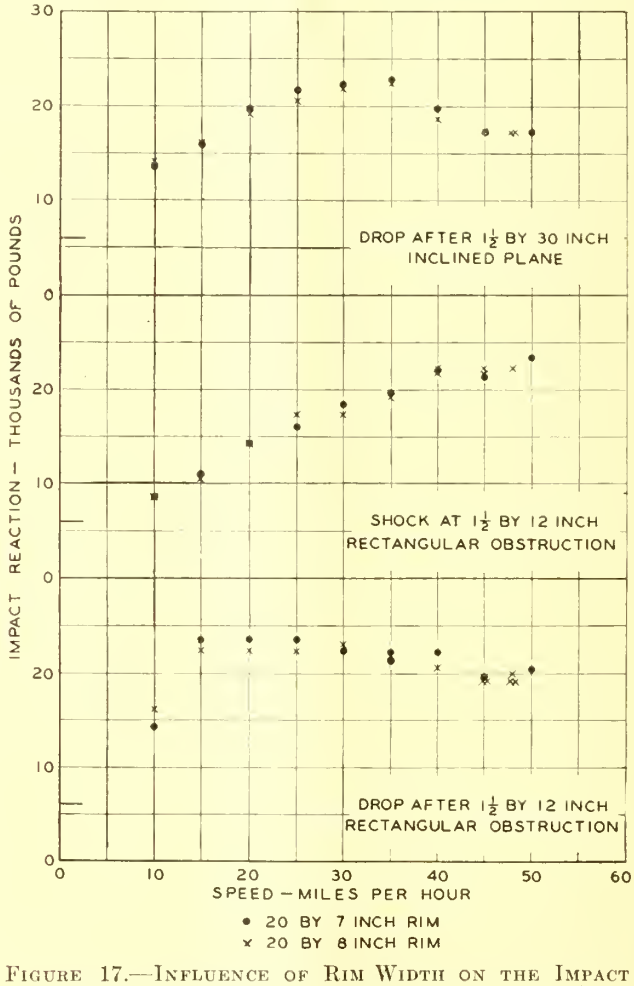
IMPACT REACTION — THOUSANDS OF POUNDS
SPEED — MILES PER HOUR
DROP AFTER 1½ BY 30 INCH INCLINED PLANE
SHOCK AT 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
DROP AFTER 1½ BY 12 INCH RECTANGULAR OBSTRUCTION
• 20 BY 7 INCH RIM
× 20 BY 8 INCH RIM

FIGURE 17.—INFLUENCE OF RIM WIDTH ON THE IMPACT REACTIONS PRODUCED BY DUAL 9.00 BY 20 INCH BALLOON TIRES. WHEEL LOAD 6,000 POUNDS; INFLATION PRESSURE, 61 POUNDS PER SQUARE INCH

relations between variable factors are adequately brought out by either computation method, the truck data were recomputed according to that used in the bus tests in order that the comparisons herein presented might be as nearly as possible on the same basis. It might also be mentioned here that the truck data were taken by an accelerometer which acted on a different principle than that which was used in the bus tests, it being the coil spring instrument, the calibration of which was described in the July, 1930, issue of Public Roads.

In Figure 19 the wheel load involved in each case was 6,000 pounds and the tire equipments were all loaded to capacity. The data for balloon tires at an inflation pressure of 61 pounds per square inch were

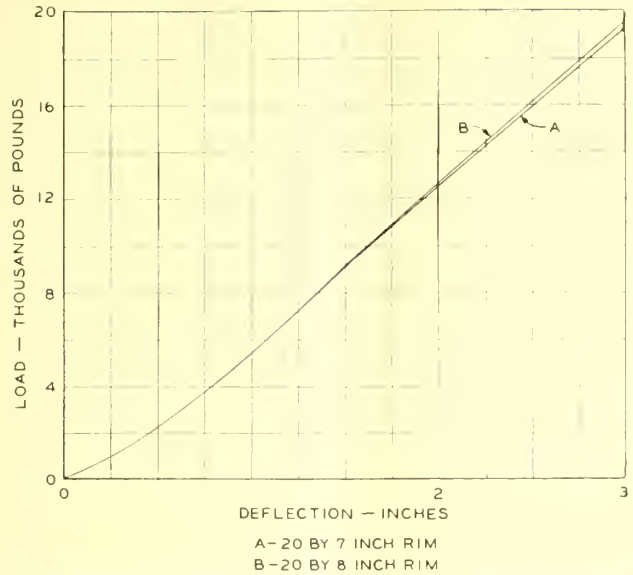

FIGURE 18.—STATIC LOAD-DEFLECTION CURVES SHOWING THE INFLUENCE OF RIM WIDTH ON A 9.00 BY 20 INCH BALLOON TIRE. INFLATION PRESSURE, 61 POUNDS PER SQUARE INCH

taken with dual 9.00 by 20 inch tires on the bus chassis. The data for high-pressure pneumatic tires at an inflation pressure of 100 pounds per square inch were taken with dual 34 by 7 inch tires on the bus chassis and with dual 38 by 7 inch tires on a 3-ton truck chassis as indicated in the figure. It is worthy of note that these data from independently conducted tests involving different vehicles and different instruments are in such close agreement. The data for new and worn solid tires were taken with dual 36 by 5 inch tires on the 3-ton truck chassis, the thickness of rubber visible above the tire flange being a little over 2 inches for the new tires and averaging a little less than 1 inch for the worn tires.

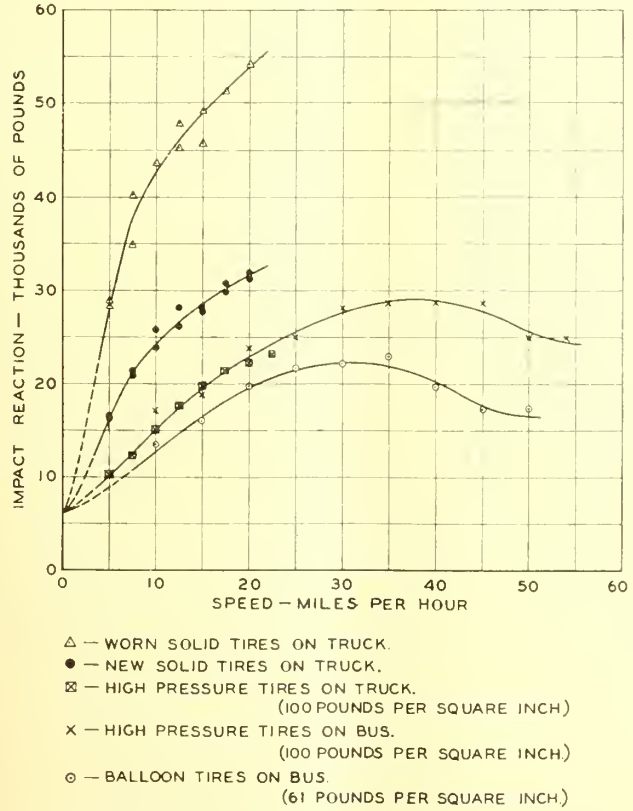
IMPACT REACTION — THOUSANDS OF POUNDS
SPEED — MILES PER HOUR
△ — WORN SOLID TIRES ON TRUCK.
● — NEW SOLID TIRES ON TRUCK.
⊠ — HIGH PRESSURE TIRES ON TRUCK. (100 POUNDS PER SQUARE INCH)
× — HIGH PRESSURE TIRES ON BUS. (100 POUNDS PER SQUARE INCH)
⊙ — BALLOON TIRES ON BUS. (61 POUNDS PER SQUARE INCH)

FIGURE 19.—COMPARISON OF DATA FROM SEPARATELY CONDUCTED TRUCK AND BUS IMPACT TESTS, SHOWING REACTIONS PRODUCED BY VARIOUS TYPES OF TIRE EQUIPMENT. PLOTTED POINTS DENOTE IMPACT REACTIONS CAUSED BY DROP AFTER 1½ BY 30 INCH INCLINED PLANE

The wheel load used in the comparison shown in Figure 20 was 7,000 pounds, the tires being loaded to capacity in each case. The balloon tires at an inflation pressure of 63 pounds per square inch were the dual 9.00 by 24 inch tires on the bus chassis and the new cushion tires were dual 36 by 7 inch on the 3-ton truck chassis. The cushion tires included three brands and the average thickness of visible rubber beyond the tire flange was $3\frac{5}{16}$ inches.

The tire equipments compared in Figure 21 were also tested at capacity loads. The data for balloon equipment were obtained with the bus chassis using dual 10.50 by 20 inch tires inflated to 75 pounds per square inch at which pressure the wheel load capacity was 9,400 pounds. The new and worn solid tires were tested at a wheel load of 8,400 pounds, using a 5-ton truck chassis. They involved dual 40 by 6 inch and single 40 by 12 inch tires of each type, the average thickness of rubber on the new solid tires being about

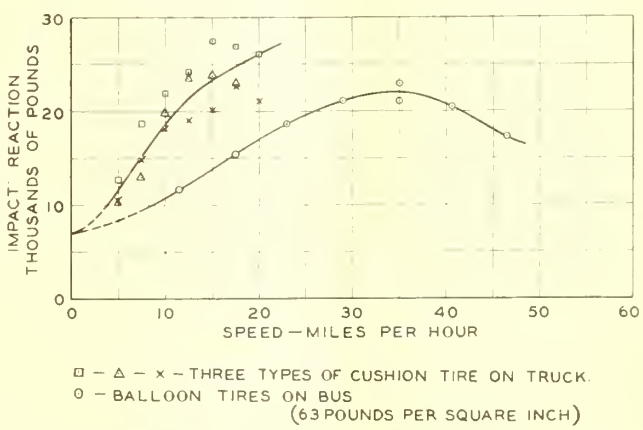
IMPACT REACTION — THOUSANDS OF POUNDS
SPEED — MILES PER HOUR
□ — △ — × — THREE TYPES OF CUSHION TIRE ON TRUCK.
○ — BALLOON TIRES ON BUS (63 POUNDS PER SQUARE INCH)

FIGURE 20.—COMPARISON OF DATA FROM SEPARATELY CONDUCTED TRUCK AND BUS IMPACT TESTS, SHOWING REACTIONS PRODUCED BY VARIOUS TYPES OF TIRE EQUIPMENT. PLOTTED POINTS DENOTE IMPACT REACTIONS CAUSED BY DROP AFTER 1½ BY 30 INCH INCLINED PLANE

two and one-half inches visible above the flange of the rim and about three-fourths inch in the case of the badly worn solid tires.

Two important facts may be drawn from the data in Figures 19, 20, and 21. First, it is evident that the data taken in the separately conducted truck and bus impact tests are compatible at least so far as drop tests are concerned. The instruments used in the two series of tests differed radically in theory of operation, yet the results obtained are directly comparable and yield reasonable relations where test conditions are selected which are subject to direct comparison.

The second important fact which may be drawn from Figures 19, 20, and 21 is that the drop impact reactions developed at the pavement surface by the wheels of busses equipped with high pressure pneumatic tires and operated at relatively high speeds are no more severe than those developed by trucks equipped with solid tires at equal or even lower wheel loads and operated at low or very moderate speeds. Further impact tests should be made, however, to determine the reactions produced under a wider range of conditions involving various types and characteristics of vehicle, tire equipment, roughness conditions and particularly

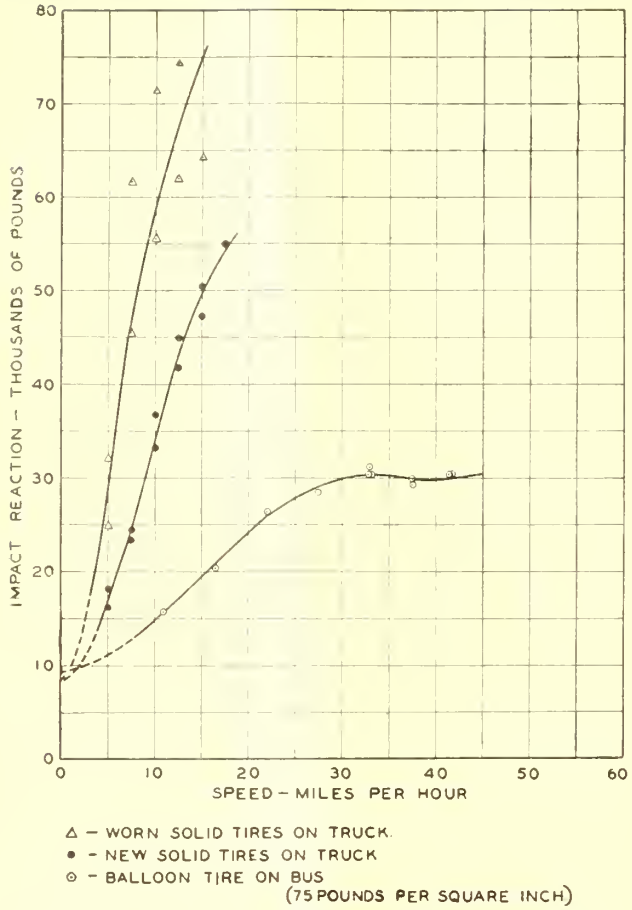
IMPACT REACTION — THOUSANDS OF POUNDS
SPEED — MILES PER HOUR
△ — WORN SOLID TIRES ON TRUCK.
• — NEW SOLID TIRES ON TRUCK
⊙ — BALLOON TIRE ON BUS
(75 POUNDS PER SQUARE INCH)

FIGURE 21.—COMPARISON OF DATA FROM SEPARATELY CONDUCTED TRUCK AND BUS IMPACT TESTS, SHOWING REACTIONS PRODUCED BY VARIOUS TYPES OF TIRE EQUIPMENT. PLOTTED POINTS DENOTE IMPACT REACTIONS PRODUCED BY DROP AFTER 1½ BY 30 INCH INCLINED PLANE

speed ranges even greater than those covered by the truck and bus tests.

TESTS MADE WITH NATURAL ROAD OBSTRUCTIONS

The artificial obstructions were intentionally made extraordinarily severe in order to expose positively the influences of the factors discussed in the preceding paragraphs. To determine what magnitudes of impact force are developed at and by actual roughness conditions occurring on the highway surface, a number of tests were made at the most severe of such "natural" obstructions as could be readily isolated and existed at locations favorable to the safe conduct of high-speed tests. Only roughness conditions which would retain their shapes and dimensions during tests were used.

In Figure 22, data are given concerning impact forces developed with the bus chassis at three natural surface roughness conditions, for three tire conditions at recommended inflation pressures. The wheel load was 6,000 pounds in each case and the maximum reactions developed under these conditions were approximately two and one-half to three times the static wheel load in the case of the dual high-pressure pneumatics, about two times static for the dual balloon and slightly less for the single balloon equipment. The conclusion which was reached in the discussion concerning artificial obstructions, that drop reactions at moderate speeds are more severe than shock reactions at a given obstruction except at high speeds, is also substantiated by these results.

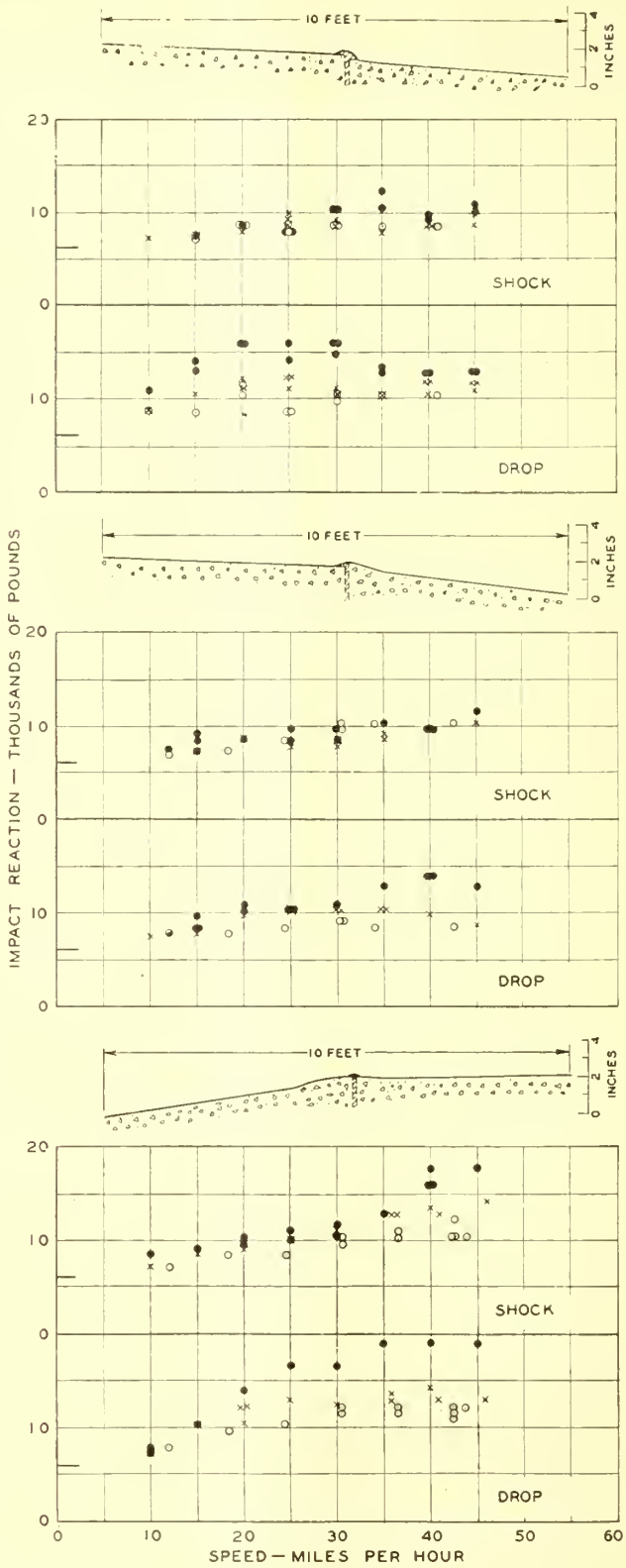
10 FEET
INCHES
SHOCK
DROP
IMPACT REACTION — THOUSANDS OF POUNDS
SPEED — MILES PER HOUR
• — 34 BY 7 INCH DUAL HIGH PRESSURE TIRES. 107 POUNDS PER SQUARE INCH
× — 9.00 BY 20 INCH DUAL BALLOON TIRES. 61 POUNDS PER SQUARE INCH.
⊙ — 12 00 BY 20 INCH SINGLE BALLOON TIRES. 82 POUNDS PER SQUARE INCH.

FIGURE 22.—IMPACT REACTIONS PRODUCED BY REAR WHEEL OF BUS CHASSIS ON ENCOUNTERING NATURAL ROAD ROUGHNESS CONDITIONS EXISTING AT TRANSVERSE JOINTS ON A CONCRETE ROAD. WHEEL LOAD, 6,000 POUNDS; DIRECTION OF TRAVEL, LEFT TO RIGHT

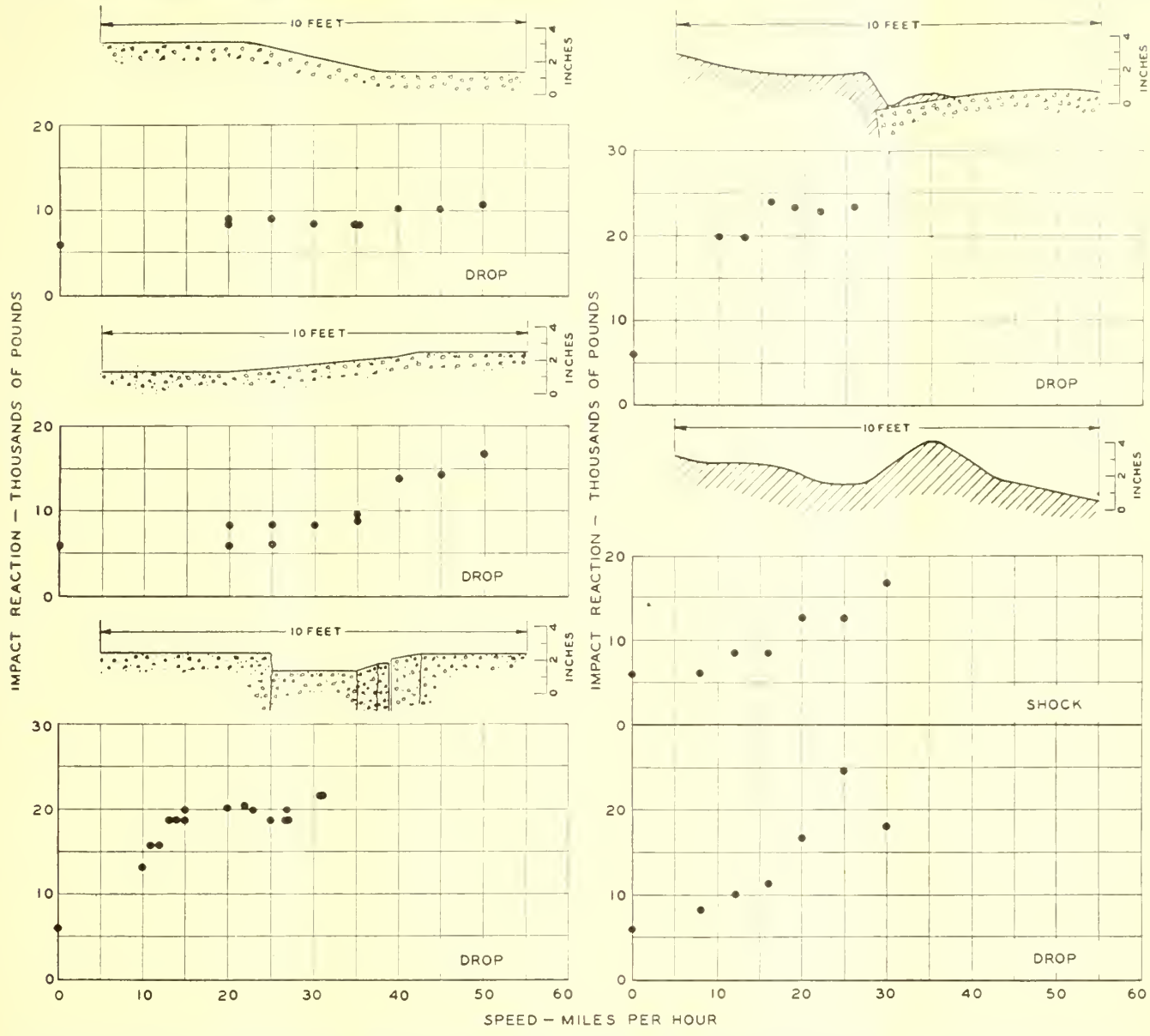

FIGURE 23.—IMPACT REACTIONS PRODUCED BY REAR WHEEL OF BUS CHASSIS ON ENCOUNTERING NATURAL ROAD ROUGHNESS CONDITIONS OF VARIOUS TYPES. WHEEL LOAD, 6,000 POUNDS; TIRE EQUIPMENT, DUAL 34 BY 7 INCH HIGH-PRESSURE; INFLATION PRESSURE 100 POUNDS PER SQUARE INCH; DIRECTION OF TRAVEL, LEFT TO RIGHT

In Figures 23 and 24, additional data concerning actual surface roughness conditions are given for dual 34 by 7 inch high-pressure tires and a wheel load of 6,000 pounds. Although high speeds were not developed for all test conditions because of safety considerations, speeds corresponding to those at which the drop reactions reached maximum values were attained. While in a few instances the reactions measured were from three to four times the static wheel load, it is significant that the surface conditions causing such reactions were all removed by maintenance within one or two weeks after the tests had been made.

It is not believed that actual roughness conditions such as were used in these tests are permitted to remain long in existence on highways even reasonably well maintained. Where they do occur they will probably be found at points where failure has already taken place and repairs are necessary. So far as these tests have indicated, the impact forces developed by balloon and high pressure tires do not reach dangerous proportions as long as the surface is reasonably smooth and designed for the wheel loads involved.

CONCLUSIONS SUMMARIZED

The data on hand were obtained under experimental conditions strictly limited as to tire sizes, tire loads, inflation pressures, rim diameters, rim widths, types of obstruction and vehicle speeds. Subject to these limitations, the following conclusions have been established:

1. Within the economic range of tire inflation pressures (roughly 10 per cent below and above standard), impact reactions vary in almost direct proportion to inflation pressures.

2. For the speeds attained during these tests (up to approximately 55 miles per hour), shock reactions increase approximately in direct proportion to speed.

3. Drop reactions reach maximum values at relatively low speeds and reactions equal to these maximum drop values are not reached under shock conditions except at relatively high operating speeds.

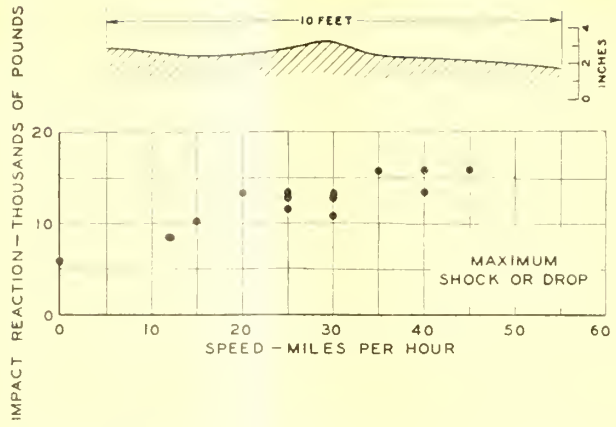

FIGURE 24.—IMPACT REACTIONS PRODUCED BY REAR WHEEL OF BUS CHASSIS ON ENCOUNTERING NATURAL ROAD ROUGHNESS CONDITIONS RESULTING FROM PATCHING CONCRETE WITH BITUMINOUS MIXTURE. WHEEL LOAD, 6,000 POUNDS; TIRE EQUIPMENT, 34 BY 7 INCH HIGH-PRESSURE; INFLATION PRESSURE, 107 POUNDS PER SQUARE INCH; DIRECTION OF TRAVEL, LEFT TO RIGHT

4. For a given natural road roughness condition, the maximum drop reaction which is obtained at a relatively low speed is not exceeded by the shock reactions, except those obtained at relatively high speeds.

5. Severe roughness conditions, such as may be occasionally found existing on actual pavements, may cause reactions as great as three or four times the static wheel load, with high-pressure tire equipment. However, it should be stated that roughness conditions which cause such excessive reactions are extreme and unusual and, in general, the artificial obstructions used in these tests are more severe than natural obstructions commonly occurring on the highway.

6. For a given cross-section of tire, either in the balloon or high-pressure type, variation in rim diameter from 20 to 24 inches has but slight influence on the magnitude of the impact reaction provided the same inflation pressure is used in each case. If standard, recommended inflation pressures are used, then the larger diameter tire causes a lower impact reaction because of the lower inflation pressure which it carries.

7. A change in rim width from 7 to 8 inches causes no appreciable change in the impact reactions of a 9.00 by 20 inch tire, the same inflation pressure being used in each case.

8. At a given inflation pressure, the variation in load from capacity to 50 per cent overload has comparatively little influence on the magnitude of the impact reactions obtained, expressed in force units. Expressed, however, as a percentage of the static load, the values obtained decrease as the load is increased.

9. For a given type, single tires cause lower reactions than dual tires whose combined capacity is equal to that of the single.

THE EFFECT OF THE DIMENSIONS OF TEST SPECIMENS ON THE FLEXURAL STRENGTH OF CONCRETE[1]

Reported by F. V. REAGEL and T. F. WILLIS, Bureau of Materials, Missouri State Highway Department

THE committee on materials of the American Association of State Highway Officials, working through a sectional committee, has been studying the flexure test of concrete in an attempt to standardize the method and apparatus for making this test. After a study of the various devices used, the sectional committee decided that in order to establish a foundation for its work, it would be necessary to design apparatus which would comply as closely as possible with the requirements of the assumptions on which the flexure formula is based. Such a design was adopted at the November, 1929, meeting of the sectional committee. The next step was to choose a standard size of specimen. To do this, it was necessary first to know what effect a variation in the dimensions of the test specimen had on the modulus of rupture, as determined with the new apparatus. The purpose of the experiment reported here was to determine this effect.

As a check on the results and in order to determine what may be expected in the way of uniformity of test results when the specially designed apparatus is used in different laboratories, this complete experiment was performed by each of four State highway laboratories, Illinois, Iowa, Missouri, and Tennessee.

EXPERIMENT COVERED WIDE VARIATION IN TEST SPECIMENS

The experiment consisted of making and testing 768 plain concrete beams. Each laboratory made three series of 64 beams, each series consisting of one beam for every possible combination of the following dimensions:

Width (inches)	4, 6, 8, and 10
Depth (inches)	4, 6, 8, and 10
Length (inches)	20, 26, 32, and 38

Throughout the experiment every effort was made to minimize any effects on the modulus of rupture caused by variables other than those being studied, viz., width and depth of test specimen, and length of span. To this end, every step was carefully outlined prior to performance of the experiment. Materials used by all laboratories were obtained from the same sources; and a man was sent to each laboratory, at the time the first series of beams was made, to assist in promoting uniformity in any way possible. From this viewpoint, it is believed that this is one of the most comprehensive cooperative concrete experiments ever undertaken.

EXPERIMENTAL PROCEDURE OUTLINED

Materials.—A sufficient quantity of each material—cement, sand and coarse aggregate—to supply all four laboratories was obtained from a single source. The cement was a standard brand of Portland cement, the entire amount of which was selected from one bin at the producer's plant, and was thoroughly mixed to insure uniformity before being divided and shipped to the different laboratories. The fine aggregate consisted of a good quality of quartz sand taken from the Missouri River at Boonville, Mo. This material was sacked from stockpile while in a damp condition and one sack out of every four, taken in the order of sacking, was shipped to a different laboratory. The coarse aggregate consisted of crushed Burlington limestone. This material was separated by screening into three sizes, 1 to 3/4 inch, 3/4 to 1/2 inch and 1/2 to 1/4 inch, and a sufficient quantity of each size was shipped to each laboratory. In this way it was assured that all batches of concrete contained an aggregate of prescribed gradation.

Each laboratory remixed its portion of each material and allowed both the fine and the coarse aggregate to become thoroughly air dry prior to the proportioning of batches for concrete.

Proportioning.—All materials were proportioned by weight into batches of the same size. Three laboratories used a batch of 2.5 cubic feet and the other (No. 2) used a 5-cubic-foot batch. All laboratories used the same water-cement ratio, 0.703.

Mixing concrete and molding specimens.—Laboratories Nos. 1, 3, and 4 used 3-cubic-foot capacity mixers and No. 2 used a 7-cubic-foot mixer. The first three washed and drained the mixer after each batch was mixed. Laboratory No. 2, using a 7-cubic-foot mixer, was unable to do this because the concrete discharge chute was so arranged that the mixer drum would not drain readily. The procedure followed in this laboratory in mixing the batches for each series of specimens, was to mix one batch, discard it, and use the next batch as the first in the series. This meant that each batch mixed in Laboratory No. 2 presumably had more mortar in it than any batch mixed in any of the other laboratories.

All specimens were molded according to a prescribed method. For the specimens 4 and 6 inches in depth the forms were filled to overflowing and the sides and ends spaded with a flat spade, by cutting the concrete away from the forms, without tamping or rodding. For the specimens 8 and 10 inches in depth the concrete was placed in the forms in two layers each of which was spaded as described above. As soon as the spading operation was finished, the tops of the specimens were struck off level with the forms by screeding with a wood straight edge, and then floated lightly with a metal trowel.

The average air temperature in the mixing room of all laboratories, and the average temperature of the freshly mixed concrete varied from 67° F. to 76° F.

Curing specimens.—All laboratories cured the beams for the first 24 hours with wet burlap and the following 27 days in moist room. The temperature and humidity records of the moist rooms during the curing period, submitted by all laboratories, showed that curing conditions were very uniform. The average curing temperatures for all laboratories fell in the range, 70° F. ±4° F.; the maximum range for any one laboratory was from 62° F. to 77° F. and these extremes occurred only for parts of a few days. The average humidities for all laboratories fell in the range, 98.5 per cent ± 1.5

[1] This article is a report of an experiment conducted by the sectional committee on flexure tests of concrete, of the American Association of State Highway Officials. The Sectional Committee is composed of the following members: F. V. Reagel, engineer of materials, Missouri State Highway Department, chairman; A. V. Bratt, engineer of materials, Massachusetts State Highway Department; E. W. Bauman, engineer of materials, Tennessee State Highway Department; V. L. Glover, engineer of materials, Illinois State Highway Department; F. G. Lang, engineer of materials, Minnesota State Highway Department; H. S. Mattimore, engineer of materials, Pennsylvania State Highway Department; Bert Myers, engineer of materials, Iowa State Highway Department; The U. S. Bureau of Public Roads. The report was submitted to and approved by the committee on materials of the American Association of State Highway Officials.

per cent; the maximum range for any one laboratory was from 97 per cent to 100 per cent.

Testing specimens.—All specimens were tested in a saturated condition at the age of 28 days. They were tested in the A. A. S. H. O. specially designed thirdpoint apparatus mounted on a universal testing machine. The speed of cross head varied from 0.05 to 0.1 inch per minute. Beams were placed in the machine so that the top faces as molded were in tension during test.

The cross-section of the beam was measured at the plane of fracture, the width to an accuracy of 0.1 inch, and the depth to an accuracy of 0.05 inch. The modulus of rupture was calculated from the formula,

$$f = \frac{S(L + 3/4\ W)}{bd^2}$$

Where

f = Modulus of rupture in pounds per square inch.
L = Total applied load in pounds.
S = Span of beam in inches.
W = Weight of beam in pounds.
b = Width of beam in inches.
d = Depth of beam in inches.

Description of apparatus.[2]—Figures 1 and 2 show different views of the apparatus used in making these tests. The test specimen is supported by the lower cross members of two free-swinging stirrups. These stirrups in turn are swung from knife-edge supports on a heavy steel member fastened across the top of the cage of the testing machine. This method of supporting the specimen eliminates any possibility of restraint of the elongation of the fibers on the tension side of the test specimen during a test.

The cross member on the lower end of one stirrup is fitted with a cylindrical segment which can rotate in a socket in a plane perpendicular to the axis of the beam and above this a knife-edge support permitting rotation in a plane parallel with the axis of the beam. The other cross member has only the knife-edge bearing. These accessories insure uniform support to a specimen during test even though the specimen is not a perfect rectangular prism.

The load-applying mechanism consists essentially of a secondary beam parallel to the axis of the specimen with a transverse plate at each end of the beam. The load is applied by the moving cross head of the machine through a cylindrical segment and socket to the center of the secondary beam and then transferred through cylindrical segments (at right angles to the first) at each end of the secondary beam and transverse plates and 2-inch rollers to the test specimen. The rollers and plates are set so that the load is applied at the one-third points of the span of the test specimen. The use of these various accessories eliminate the possibility of inducing torsional stress in the specimen, a condition often found where a rigid load-applying apparatus is used. The various parts of the load-applying mechanism are joined by flexible springs which permit the entire mechanism to be handled as a unit when placing a specimen in position.

It is believed that this apparatus successfully performs its intended function, viz., to insure that the specimen is subjected only to coplanar forces perpendicular to the axis of the beam and that there is no restraint to the deformation of the fibers of the specimen.

In the tests under discussion the apparatus appeared to work perfectly. All the devices which were installed to insure proper stressing of the specimen functioned as intended. While the apparatus was a little cumbersome and each test required considerable time, this was largely because the machine had to be built to accommodate the various sizes of beams used in this experiment. An apparatus similarly constructed but intended for use on a single size of specimen would be much simpler to operate.

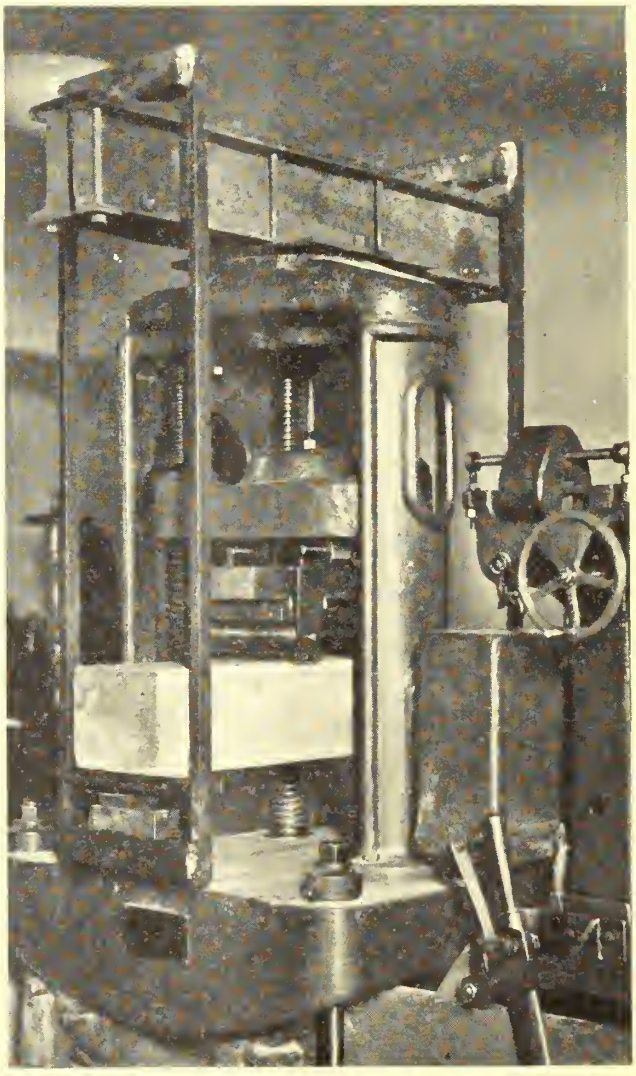

FIGURE 1.—GENERAL VIEW OF APPARATUS FOR FLEXURE TESTS OF CONCRETE BEAMS, SHOWING SPECIMEN IN PLACE, READY TO BE TESTED

UNIFORMITY OF MATERIALS AND CONCRETE TESTED

The tests performed by the four laboratories on the materials used in this experiment are shown in Tables 1, 2, and 3. In general the data are very consistent and show that the attempt to provide similar materials for all the laboratories was successful.

The tabulation given below shows the consistency of the concrete used by the different laboratories.

Laboratory No.	1	2	3	4
Average slump (inches)	1.7	1.1	2.4	0.8
Maximum slump (inches)	2.2	1.8	2.7	0.9
Minimum slump (inches)	1.5	0.9	2.1	0.5

[2] This apparatus was designed for the committee by F. N. Wray, research assistant, Missouri State Highway Department, and embodies several suggestions made by Messrs. Bert Myers and A. E. Stoddard of the Iowa State Highway Department and T. F. Willis of the Missouri State Highway Department.

TABLE 1.—*Characteristics of cement used in flexure tests of concrete beams*

Laboratory No.	Sample No.	Tensile strength (pounds per square inch) 7 days	Tensile strength (pounds per square inch) 28 days	Per cent water at normal consistency	Time of set (minutes) Initial	Time of set (minutes) Final	Fineness: Per cent retained on No. 200 screen	Soundness
1	1	395	461	23.6	190	370	6.8	Satisfactory.
	2	397	478	23.6	160	355	6.1	Do.
	3	361	423	23.6	180	370	6.3	Do.
Average		384	454	23.6	177	365	6.4	Do.
2	1	388	478	24.0	185	270	5.8	Do.
	2	377	452	24.0	185	260	5.9	Do.
	3	378	428	24.5	205	335	6.0	Do.
Average		381	452	24.2	192	288	5.9	Do.
3	1	367	520	23.5	180	360	7.5	Do.
	2	412	492	24.0	205	385	6.7	Do.
	3	408	447	24.0	210	405	6.1	Do.
Average		396	486	23.8	198	383	6.8	Do.
4	1	417	485	23.0	227	410	5.4	Do.
	2	425	473	23.0	230	400	6.0	Do.
	3	405	465	23.0	234	403	6.0	Do.
Average		416	474	23.0	230	404	5.8	Do.
Average of all laboratories		394	466	23.6	204	360	6.2	Do.

TABLE 2.—*Characteristics of fine aggregate used in flexure tests of concrete beams*

Laboratory No.	Sample No.	Weight per cubic foot (dry-rodded)	Per cent void space	Mechanical analysis, per cent passing square mesh screens No. 4	No. 8	No. 14	No. 28	No. 48	No. 100	Strength ratio [1] 7 days	28 days
1	[2] 1	114	31	97.2	88.6	69.7	34.4	11.6	2.9	1.22	1.08
	2	114	31	97.0	85.1	64.5	30.3	10.8	1.0	1.23	1.07
Average		114	31	97.2	88.0	68.8	33.4	11.5	2.4	1.22	1.08
2	1	115	30	95.5	84.6	65.5	33.7	10.0	1.8	1.09	1.12
	2	115	30	96.0	87.0	70.0	37.0	12.0	2.0	1.30	1.18
	3	115	30	96.0	88.0	70.0	39.0	12.0	2.0	1.14	1.14
Average		115	30	95.8	86.5	68.5	36.6	11.3	1.9	1.18	1.15
3	1	114	31	96	84	62	29	9	1	1.31	.95
	2	113	32	98	86	66	30	9	1	1.15	1.07
	3	114	31	95	83	60	26	8	1	1.10	1.17
Average		114	31	96.3	84.3	62.7	28.3	8.7	1.0	1.19	1.06
4	1	[3] 102	[3] 38	97.3	89.4	70.2	31.9	11.5	1.8	1.23	1.16
	2	[3] 102	[3] 39	96.3	86.6	65.4	30.5	9.7	1.1	1.26	1.17
	3	[3] 102	[3] 39	96.7	86.3	66.4	30.2	10.8	1.7	1.39	1.21
Average		[3] 102	[3] 39	96.8	87.4	67.3	30.9	10.7	1.5	1.29	1.18
Average of all laboratories		114	31	96.5	86.5	66.8	32.3	10.7	1.7	1.22	1.12

[1] In terms of values given by standard Ottawa sand.
[2] Average of test results on 5 samples.
[3] Values determined for damp loose volumes—excluded from average.

NOTE.—Specific gravity varied from 2.63 to 2.65 but average from each laboratory gave 2.64.

MEASUREMENTS OF MODULUS OF RUPTURE SHOW SIMILAR UNIFORMITY

In table 4 are given the means of the variations of the individual tests of the three series from the average of the three series of tests conducted in each of the four laboratories. The values in the table are expressed as percentages of the average moduli of rupture for the four series. The table is not complete in that it does not give values for all the different sizes of beams, but the values given are representative of the entire group. For each size of beam the average of the mean variations for the four laboratories is tabulated; and for each laboratory the average of the mean variations for all sizes is also given.

TABLE 3.—*Characteristics of coarse aggregate used in flexure tests of concrete beams*

Laboratory No.	Sample No.	Weight per cubic foot (dry-rodded)	Per cent void space	Apparent specific gravity	Per cent absorption in 30 minutes	Per cent passing 1½-inch square screen	Per cent passing ¾-inch square screen	Per cent passing ⅜-inch square screen
1	[1] 1	91.8	45.2	2.68	0.2	100	76.2	13.4
2	1	95.1	43.3	2.69	0.4	100	72.0	14.0
	2	96.0	43.0	2.69	0.3	100	73.2	15.3
	3	95.6	43.1	2.69	0.3	100	76.5	13.5
Average		95.6	43.1	2.69	0.3	100	73.9	14.3
3	1	92.7	44.1	2.66	0.5	100	76.0	15.0
	2	90.4	45.7	2.67	0.4	100	77.0	15.0
	3	90.0	46.1	2.67	0.4	100	77.0	14.0
Average		91.0	45.3	2.67	0.4	100	76.7	14.7
4	1	92.0	45.7	2.71	0.2	100	70.6	9.2
	2	92.0	45.5	2.70	0.1	100	70.9	8.0
	3	91.0	46.3	2.71	0.1	100	63.6	8.5
Average		91.7	45.8	2.71	0.1	100	68.2	8.6
Average of all laboratories		92.5	44.9	2.69	0.25	100	73.8	12.8

[1] Average of test results on 4 samples.

It will be observed that, in general, the larger sizes of beams showed a lower mean variation. This may be due to the effect of coarse aggregate, which in this experiment was of 1-inch maximum size. The difference in mean variation of results for the large and small beams is not entirely consistent, nor is its magnitude sufficient to warrant the conclusion that the smaller sizes of beams should not be used with 1-inch coarse aggregate. There is, however, an indication that the larger sizes given more consistent results.

The averages of the mean deviations for each laboratory range from 3.16 per cent to 4.40 per cent with a grand average of 3.86 per cent. This is considered better than ordinary for concrete testing.

Table 5 shows the percentage variation of the average of each laboratory's results from the average of the four laboratories for several sizes of specimen; and appended to the table are average moduli of rupture for all tests performed by each laboratory.

In either the values for any one laboratory or the averages for the four laboratories there is no consistent relation between percentage variation and size

TABLE 4.—*Mean percentage variations of individual test results of three series from the average values of modulus of rupture for all series*

Dimensions of beam [1]	Mean variation of individual test result: Laboratory No. 1	Laboratory No. 2	Laboratory No. 3	Laboratory No. 4	Average of all laboratories
Inches	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
4 by 4 by 18	4.07	7.31	3.92	6.24	5.38
4 by 4 by 24	1.75	7.64	5.58	3.62	4.65
4 by 4 by 30	3.10	5.42	7.90	3.36	4.94
4 by 4 by 36	3.53	6.43	5.50	5.49	5.24
6 by 6 by 18	0.71	3.26	4.13	4.96	3.27
6 by 6 by 24	3.18	6.44	2.17	4.98	4.19
6 by 6 by 30	2.30	3.22	2.17	1.45	2.28
6 by 6 by 36	4.43	3.49	5.32	5.07	4.58
8 by 8 by 18	2.01	3.23	4.03	5.97	3.81
8 by 8 by 24	4.04	4.64	3.58	0.76	3.26
8 by 8 by 30	4.56	4.00	1.97	2.86	3.35
8 by 8 by 36	2.61	3.99	3.07	7.14	4.20
10 by 10 by 18	2.31	3.71	1.34	2.57	2.48
10 by 10 by 24	2.64	2.76	4.32	2.92	3.16
10 by 10 by 30	5.84	1.95	1.35	4.14	3.32
10 by 10 by 36	3.43	2.92	3.69	4.40	3.61
Average of all sizes	3.16	4.40	3.75	4.12	3.86

[1] Dimensions are given in depth, width and length of span. For total length of beam, add 2 inches.

of beam. This indicates that the beam size did not affect the uniformity of results obtained by the different laboratories; in other words the different laboratories check each other as well with one size of beam as with another.

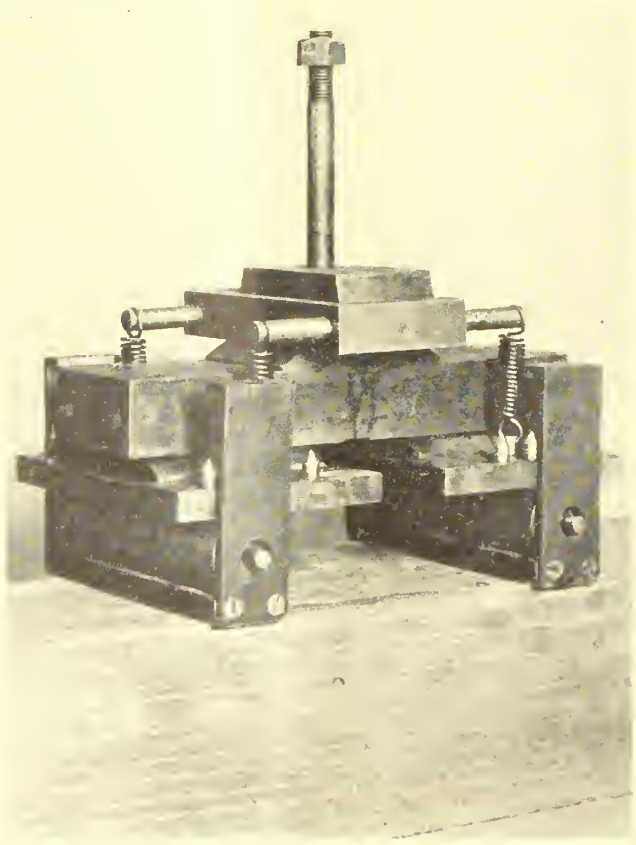

FIGURE 2.—APPARATUS FOR THIRD-POINT APPLICATION OF LOAD TO CONCRETE BEAMS

TABLE 5.—*Percentage variation of average values of modulus of rupture for each laboratory from average of four laboratories*

Dimensions of beam	Variation of average modulus of rupture				Average of all laboratories
	Laboratory No. 1	Laboratory No. 2	Laboratory No. 3	Laboratory No. 4	
Inches	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
4 by 4 by 18	0.46	12.24	11.44	1.49	6.41
6 by 4 by 18	.22	5.23	7.69	2.21	3.68
8 by 4 by 18	1.26	5.51	1.49	2.41	2.67
10 by 4 by 18	4.25	4.14	1.73	1.84	2.99
4 by 6 by 24	3.09	5.44	4.20	1.73	3.62
6 by 6 by 24	5.65	5.77	2.04	2.28	3.94
8 by 6 by 24	1.91	9.42	1.19	6.32	4.71
10 by 6 by 24	5.80	8.88	.11	3.20	4.50
4 by 8 by 30	2.60	5.20	.65	2.21	2.67
6 by 8 by 30	.49	3.62	4.00	1.00	2.28
8 by 8 by 30	9.34	7.67	1.92	.26	4.80
10 by 8 by 30	4.41	7.30	3.27	5.16	5.04
4 by 10 by 36	.68	4.65	1.37	4.10	2.70
6 by 10 by 36	5.28	2.51	5.81	2.90	4.12
8 by 10 by 36	6.06	7.51	4.34	5.66	5.89
10 by 10 by 36	5.82	6.34	6.34	6.72	6.30
Average of all sizes	3.58	6.34	3.56	3.09	4.14
Average modulus of rupture for all tests	787	857	800	804	812

The average percentage variations are shown in the next to the last horizontal line. The average variations for the results from three of the laboratories are approximately the same, but that of laboratory No. 2 is nearly twice as great. The average of all results for laboratory No. 2 is considerably above the averages for the other laboratories, and this is consistently true of their individual results. In other words the results were consistent within themselves, but for some reason were higher than the corresponding results of the other laboratories. The record of molding, curing, and testing specimens reveals only one radical difference between the procedure used by this laboratory and that used by the others, viz., that they mixed a double batch, did not wash or drain the mixer after mixing each batch, and therefore probably had a higher mortar content in each batch than did the other laboratories. Only experiment can tell whether this was the cause of the higher test results.

In spite of this the grand average for all laboratories is only 4.14 per cent, which is approximately the same as the four laboratories' average mean percentage variation of individual tests from the average of three series. In other words, it appears that the degree of uniformity between the average results obtained in four different laboratories was as good as the uniformity between the three series of tests run in any one laboratory. This indicates that by maintaining properly standardized

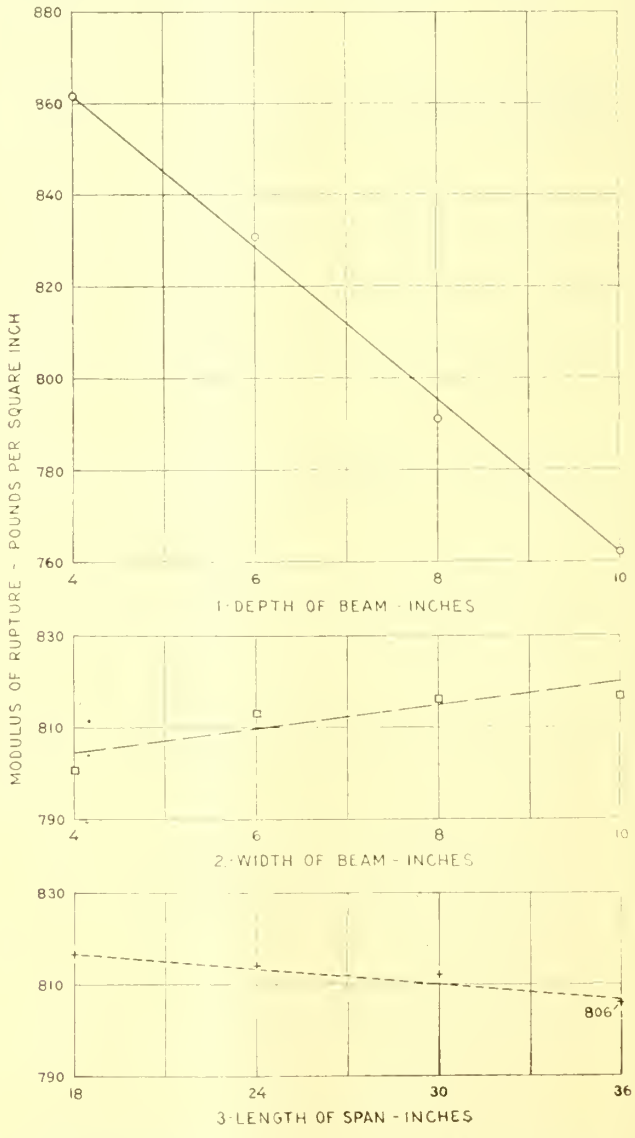

FIGURE 3.—EFFECT OF DEPTH OF BEAM, WIDTH OF BEAM, AND LENGTH OF SPAN ON MODULUS OF RUPTURE OF CONCRETE TEST SPECIMENS, AS SHOWN BY AVERAGES OF RESULTS FROM FOUR LABORATORIES

procedures and using the same materials, different laboratories can check each other within reasonable limits. The results are particularly gratifying in view of the time and expense gone to in preparing for this experiment.

DATA ANALYZED AND DISCUSSED

The results of the tests of modulus of rupture are given in Table 6 and Figures 3 to 9. Table 6 contains values of modulus of rupture for all the tests. The tabulated values which are not marked as averages are themselves averages of three tests made on different days. It will be observed that a complete system of averaging has been carried out. The different types of average which are given may be described as follows:

For a given laboratory, a given span, and a given width:
The average value given by all four depths.
For a given laboratory, a given span, and a given depth:
The average value given by all four widths.
For a given laboratory and a given span:
The average value given by all combinations of width and depth.
For all four laboratories and a given span:
The average value given by each combination of width and depth.
For all four laboratories, a given span, and a given width:
The average value given by all four depths.
For all four laboratories, a given span, and a given depth:
The average value given by all four widths.
For all four laboratories and a given span:
The average value given by all combinations of width and depth.
For a given laboratory, all four spans, and a given width:
The average value given by all four depths.
For a given laboratory, all four spans, and a given depth:
The average value given by all four widths.
For a given laboratory and all four spans:
The average value given by all combinations of width and depth.[3]
For all four laboratories, all four spans, and a given width:
The average value given by all four depths.

[3] This average value appears twice in Table 6.

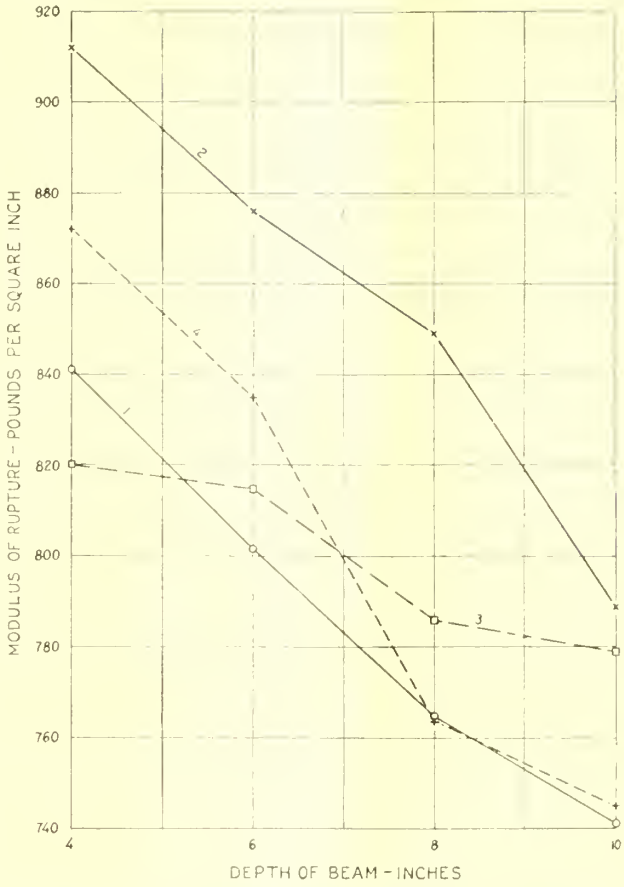

FIGURE 4.—EFFECT OF DEPTH OF BEAM ON THE MODULUS OF RUPTURE OF CONCRETE TEST SPECIMENS. AVERAGE VALUES FOR ALL WIDTHS OF BEAM AND ALL LENGTHS OF SPAN ARE PLOTTED SEPARATELY FOR EACH LABORATORY

TABLE 6.—*Results of tests to determine the effect of the dimensions of test specimen on the flexural strength of concrete. Values of the modulus of rupture, in pounds per square inch, are tabulated according to width and depth of beam and length of span*

Laboratory No.	Width of beam	18-inch span: Depth of beam in inches 4	6	8	10	Average	24-inch span: Depth of beam in inches 4	6	8	10	Average	30-inch span: Depth of beam in inches 4	6	8	10	Average	36-inch span: Depth of beam in inches 4	6	8	10	Average	Average of all spans and all depths	Depth of beam	Average of all spans and all widths
	Inches																						*Inches*	
1	4	878	777	754	701	778	876	784	752	752	791	828	806	749	772	789	831	799	797	736	791	787	4	841
	6	859	842	817	734	813	850	785	755	729	780	811	769	797	771	787	825	790	760	718	773	788	6	802
	8	860	825	731	743	790	813	823	768	721	781	831	823	709	766	782	844	803	766	713	782	784	8	765
	10	833	824	791	793	810	858	796	751	731	784	850	792	759	748	787	799	786	775	728	772	788	10	741
Average		858	817	773	743	798	850	797	757	733	784	830	798	754	764	786	825	795	775	724	780	787		787
2	4	981	845	843	786	864	899	853	799	777	832	941	848	809	719	829	856	857	814	765	824	837	4	912
	6	906	849	909	787	863	918	880	844	800	860	918	817	830	785	838	963	867	860	777	867	857	6	876
	8	919	902	857	775	863	890	918	890	834	884	927	900	842	803	868	913	888	860	816	869	871	8	849
	10	906	890	865	774	859	904	920	867	798	872	868	883	852	807	852	888	895	862	822	867	864	10	789
Average		928	871	868	780	862	903	893	850	802	862	913	862	833	778	847	905	877	849	795	857	857		857
3	4	774	750	771	761	764	819	775	758	791	786	810	816	764	742	783	777	781	766	721	761	773	4	820
	6	800	799	769	750	779	801	815	764	778	789	812	817	869	786	795	795	833	810	802	811	793	6	815
	8	858	829	802	813	825	861	829	746	810	812	837	850	797	795	820	842	850	792	792	819	817	8	786
	10	855	804	867	770	824	849	844	783	770	812	822	825	820	765	808	811	838	794	822	816	815	10	779
Average		822	794	802	773	798	832	816	762	787	799	820	826	787	772	801	806	825	791	784	802	800		800
4	4	861	872	784	713	807	893	823	797	751	816	892	880	752	767	823	879	779	725	701	771	804	4	872
	6	880	928	794	759	840	892	850	774	784	825	863	792	809	761	806	895	808	727	736	792	816	6	835
	8	850	829	701	745	781	855	786	747	735	781	855	864	780	762	815	870	813	780	716	795	793	8	764
	10	886	854	755	737	808	887	818	774	763	810	852	852	753	766	805	837	808	766	721	783	802	10	745
Average		869	871	759	738	809	882	819	773	758	808	866	847	773	764	812	870	802	749	719	785	804		804
Average of four laboratories	4	874	811	788	741	803	872	809	777	768	807	868	837	769	750	806	836	804	776	731	787	801	4	862
	6	861	854	822	757	823	865	832	784	773	814	851	798	801	776	806	870	824	790	758	810	813	6	832
	8	871	846	773	769	815	855	839	788	775	814	863	859	782	781	821	868	838	799	759	816	816	8	791
	10	870	843	819	769	825	875	845	794	765	820	848	838	794	771	813	834	832	799	773	810	817	10	763
Average		869	838	801	759	817	867	831	786	771	814	857	833	787	769	812	852	824	791	755	806	812		812

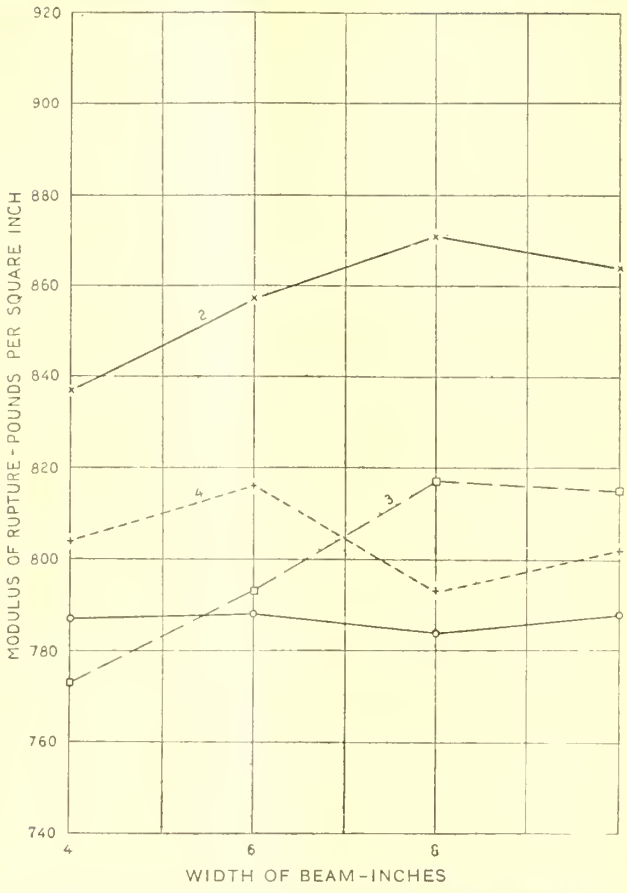

FIGURE 5.—EFFECT OF WIDTH OF BEAM ON THE MODULUS OF RUPTURE OF CONCRETE TEST SPECIMENS. AVERAGE VALUES FOR ALL DEPTHS OF BEAM AND ALL LENGTHS OF SPAN ARE PLOTTED SEPARATELY FOR EACH LABORATORY

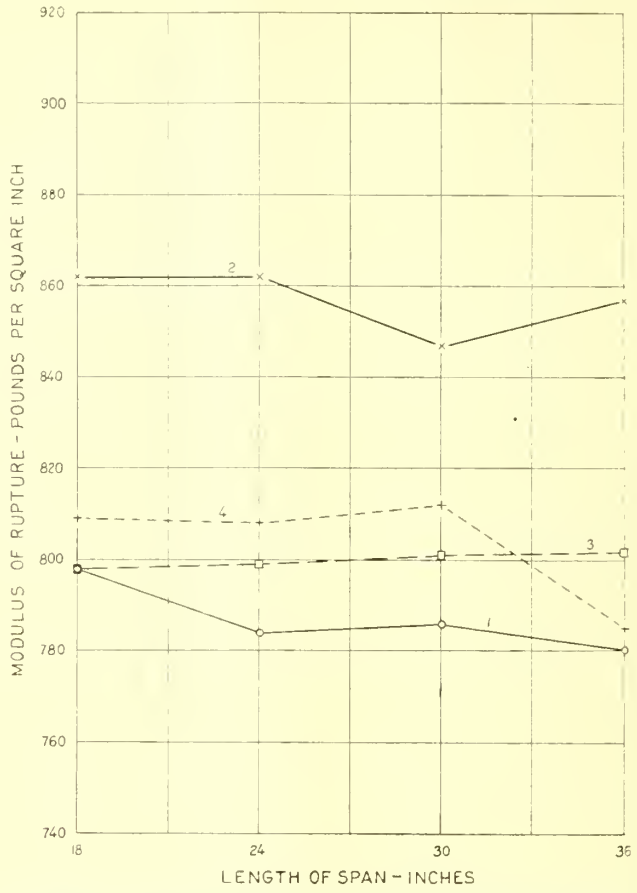

FIGURE 6.—EFFECT OF LENGTH OF SPAN ON THE MODULUS OF RUPTURE OF CONCRETE TEST SPECIMENS. AVERAGE VALUES FOR ALL DEPTHS AND WIDTHS OF BEAM ARE PLOTTED SEPARATELY FOR EACH LABORATORY

For all four laboratories, all four spans, and a given depth: The average value given by all four widths.

The grand average value of all test results, i. e., all four laboratories, and all combinations of span length, width of beam and depth of beam.[4]

In Figure 3 are given the final average curves showing the variation of modulus of rupture with depth of beam, width of beam and length of span. In the case of depth each plotted point represents the average of all test results from the four laboratories and all combinations of width of beam and length of span. Similar averages are represented by the plotted points on the other two graphs. In Figures 4, 5, and 6, the average results are plotted separately for each laboratory. For example, in Figure 4 each curve represents an average of all the data from a given laboratory, and each plotted point represents the average modulus of rupture given by all tests, in a given laboratory, of beams of a given depth, regardless of width of beam and length of span. The curves of Figures 4, 5, and 6 bring out clearly the fact, previously noted, that the test results obtained in Laboratory No. 2 were consistently higher than those obtained in the other three laboratories.

In Figures 7, 8, and 9 data from the four laboratories are averaged, different values of the three dimensions, width, depth, and length of span, being represented by separate curves and panels. Thus, in Figure 7 each panel contains data relative to a given length of span, averaged for the four laboratories; and each curve shows average results of tests of beams of a given width. Each plotted point, therefore, represents the average modulus of rupture for the tests, in all four laboratories, of all beams having a given depth, a given width, and a given length of span. Average curves are drawn to illustrate the trend of the variation for all widths. Figures 8 and 9 are constructed in a similar manner to show the effect of width of beam and length of span.

MODULUS VARIES INVERSELY WITH DEPTH OF SPECIMEN

Curve No. 1 in Figure 3, which is based on the averages of all the results from all the laboratories, shows that there is practically a straight line relationship between depth of specimen and modulus of rupture, the modulus of rupture decreasing 1.9 per cent for each inch increase in the depth of specimen.[5] Increasing the depth of specimen from 4 to 10 inches resulted in decreasing the average modulus of rupture from 862 to 763 pounds per square inch, a net reduction of 100 pounds per square inch for a 6-inch increase in depth of specimen.

The curves of Figure 4 show that the results of each of the four laboratories, considered individually, have the same qualitative relationship; while the curves of Figure 7 indicate that the relationship is approximately the same regardless of width of specimen or length of span.

EFFECT OF WIDTH NEGLIGIBLE

Figures 3, 5, and 8 show the effect of width of test specimen on the modulus of rupture. Curve No. 2 is Figure 3, based on the averages of results of all tests, shows that the modulus of rupture increased very slightly

[4] This arrangement value appears twice in Table 6.

[5] This confirms qualitatively the results of tests by the Portland Cement Association. See Report of Director of Research, November, 1928, p. 174.

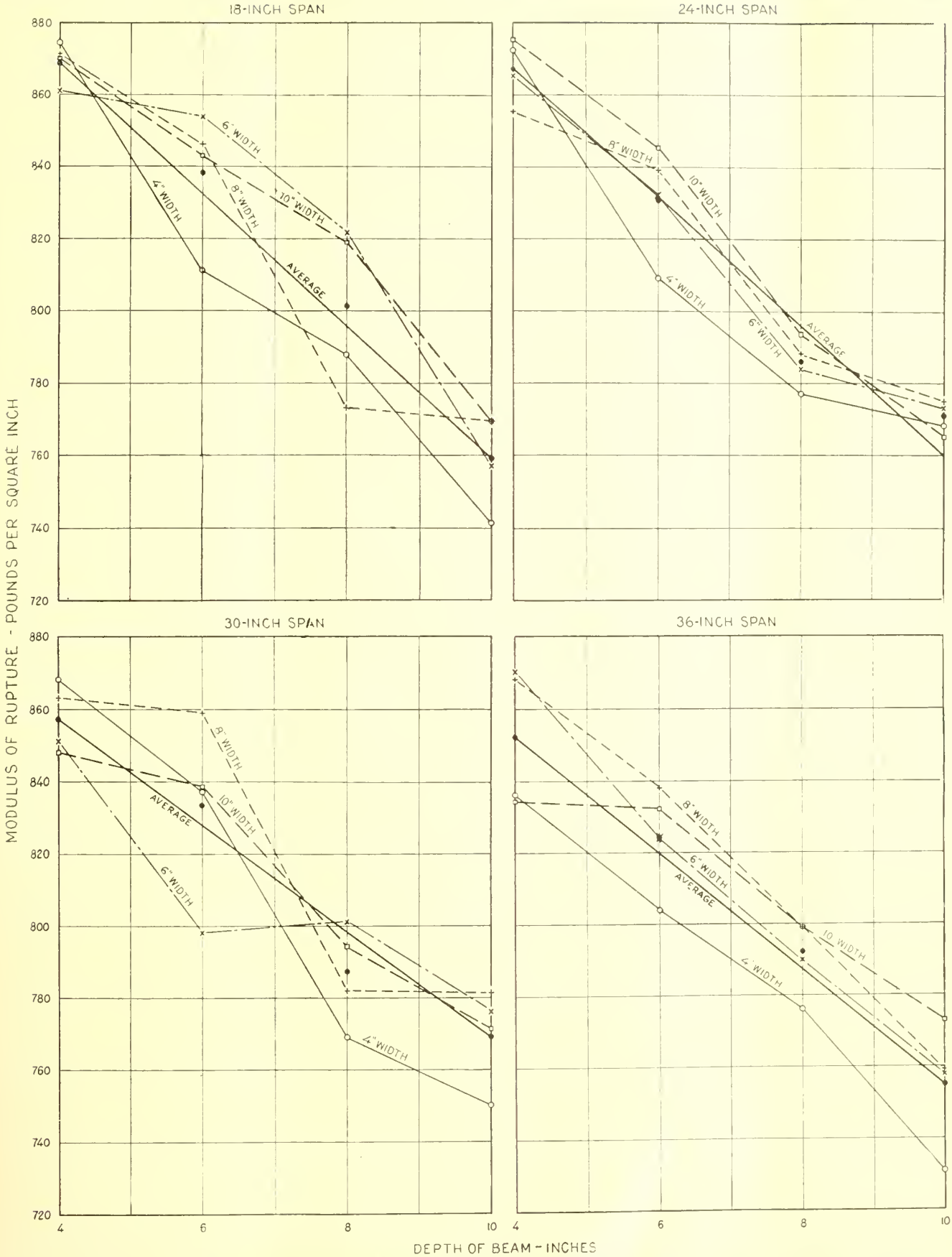

FIGURE 7.—EFFECT OF DEPTH OF BEAM ON THE MODULUS OF RUPTURE OF CONCRETE TEST SPECIMENS. VALUES OF MODULUS OF RUPTURE ARE AVERAGED FOR ALL FOUR LABORATORIES, BUT ARE PLOTTED SEPARATELY FOR EACH WIDTH OF BEAM AND EACH LENGTH OF SPAN

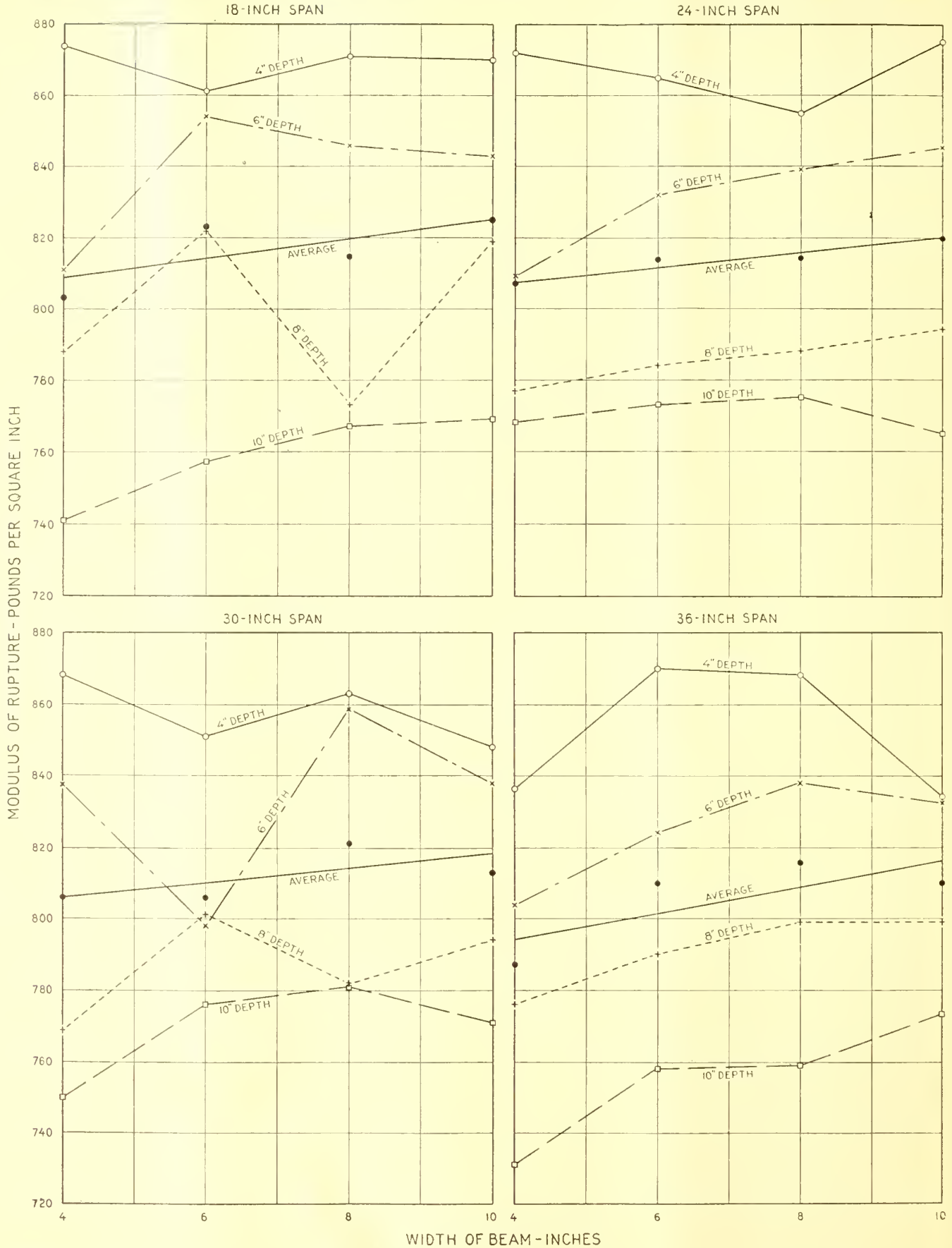

FIGURE 8.—EFFECT OF WIDTH OF BEAM ON THE MODULUS OF RUPTURE OF CONCRETE TEST SPECIMENS. VALUES OF MODULUS OF RUPTURE ARE AVERAGED FOR ALL FOUR LABORATORIES, BUT ARE PLOTTED SEPARATELY FOR EACH DEPTH OF BEAM AND EACH LENGTH OF SPAN

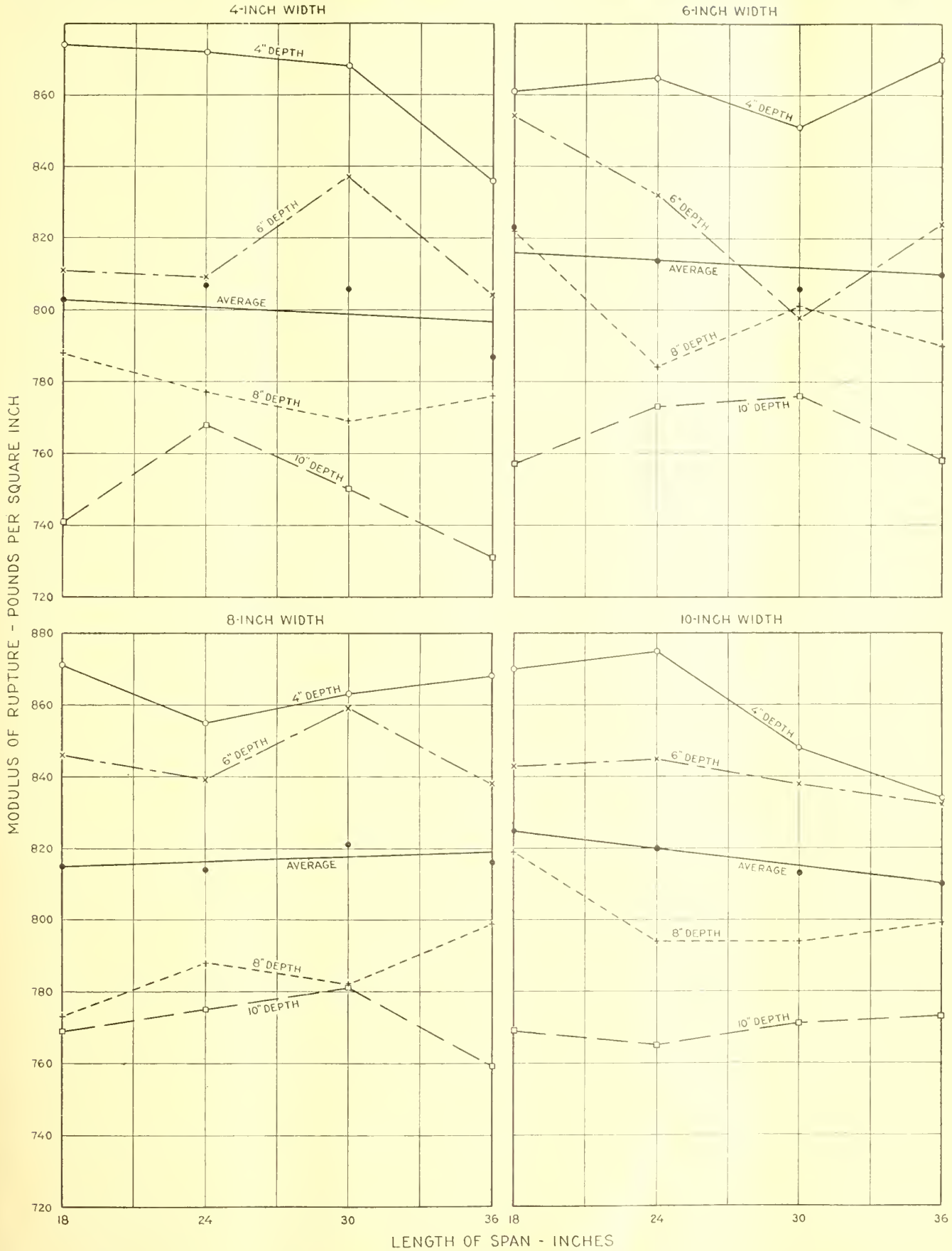

FIGURE 9.—EFFECT OF LENGTH OF SPAN ON THE MODULUS OF RUPTURE OF CONCRETE TEST SPECIMENS. VALUES OF MODULUS OF RUPTURE ARE AVERAGED FOR ALL FOUR LABORATORIES, BUT ARE PLOTTED SEPARATELY FOR EACH WIDTH AND EACH DEPTH OF BEAM

as the width of specimen increased. The average modulus of rupture increased approximately 16 pounds per square inch when the width of beam was increased from 4 to 10 inches. The greatest increase is between the 4 and 6 inch widths. It is possible that the results for the 4-inch width are slightly low on account of the difficulty of molding specimens of this width properly.

It may be noted that this increase of 16 pounds per square inch is but 2 per cent of the average value of modulus of rupture, 801, for beams of 4-inch width. An examination of Table 4 will show that the mean percentage variation of individual test results from the average of the three series is generally much greater than 2 per cent. There are individual cases where it is less than 1 per cent, but the average for each laboratory is greater than 3 per cent, and the average for all tests is 3.86 per cent. Similarly, in Table 5, it is shown that the variation of the average for one laboratory varies from the average of four laboratories by values running up to 9.42 per cent, with an average for all tests of 4.14. These facts make it apparent that the indicated variation in modulus of rupture due to changes in the width of beam is less than the probable variation due to influences not in the control of the experimenter.

Reference to Figure 5 shows that the trend of the curve in Figure 3 is due to the results of laboratories Nos. 2 and 3, and that average curves for the other two laboratories would approximate horizontal lines. It is, therefore, doubtful that increasing the width of specimen causes an increase in the modulus of rupture. For all practical purposes the average results of all four laboratories indicate that varying the width of the specimen had no effect on the modulus of rupture.

NO MARKED EFFECT DUE TO VARIATION IN SPAN LENGTH

Figures 3, 6, and 9 show the effect of the length of span. Curve No. 3 in Figure 3, based on the averages of results of all tests, shows that the modulus of rupture decreased very slightly as the span length was increased. The average modulus of rupture decreased approximately 9 pounds per square inch when the length of span was increased from 18 inches to 36 inches. Figure 6 shows the average results for each laboratory. It will be noted that two of the laboratories (Nos. 2 and 3) show practically no effect from varying span, one of them (No. 1) showed a decrease of 18 pounds per square inch, and one of them (No. 4) showed a decrease of 25 pounds per square inch. These decreases are very small, are well within the limit of error of the averages, and are not corroborated by all the laboratories. This would indicate that the length of span had no appreciable effect on the modulus of rupture.[6]

CONCLUSIONS SUMMARIZED

The following conclusions, derived from the results of this experiment, are naturally restricted in their application to apparatus in which the specimen is loaded at the third points.

1. Variation in depth of specimen causes sufficient variation in the modulus of rupture that it is necessary to standardize this dimension.
2. Variation in width of specimen has a negligible effect on the modulus of rupture. This dimension might have more influence if other sizes of coarse aggregate were used.
3. Variation in the length of span has a negligible effect on the modulus of rupture.
4. Different laboratories using similar materials and methods of test can check each other with a variation not exceeding 5 per cent.

[6] This confirms, qualitatively, the results of tests by the Portland Cement Association. See Report of Director of Research, November, 1928, p. 174.

MEETING OF THE JOINT COMMITTEE ON CONCRETE AND REINFORCED CONCRETE

THE Joint Committee on Standard Specifications for Concrete and Reinforced Concrete met in the rooms of the Western Society of Engineers, Chicago, Ill., on February 27 and 28, 1931. This committee, which is made up of five representatives each of the American Society of Civil Engineers, the American Society for Testing Materials, the American Railway Engineering Association, the Portland Cement Association, and the American Concrete Institute, is a reorganization of the committee that made reports on concrete specifications in 1921 and 1924.

A number of developments since the 1924 report make certain changes in the specification desirable. Among the advances which have been made relating to concrete as a material in recent years, the most significant are the developments in cement, use of ready-mixed concrete, and a more widespread understanding of the design of mixtures with increasing attention to field control.

In the field of design, an outstanding development which should be covered in future reports of the committee, is the tendency toward the greater use of rigid frame construction.

All these developments are to be given consideration by the reorganized committee in addition to a general study of the 1924 report with a view to improving its presentation and widening its scope.

The committee proposes in its report to separate portions which are in the nature of specifications from those which are in the nature of recommended practice. It is planned also to add as an appendix the technical data upon which the recommendations of the report are based. The committee is considering the necessity of distinguishing between the requirements for so-called outdoor concrete and concrete in locations such as heated buildings not exposed to the effect of weather. Water-tightness as an element of durability will be recognized as one of the essentials of concrete for outdoor exposure. The necessity for this has been brought about by the recent development of generally higher strengths in Portland cements. Following the practice which has become quite general of designing concrete for a given strength requirement, it is possible with these newer cements that mixtures will result which are too lean for proper durability. Some limitation will, therefore, be placed upon the cement content to avoid this difficulty.

In the field of design of reinforced concrete, the committee proposes to present the recommendations in regard to moment coefficients by putting primary emphasis on the general case of unequal spans, thus reversing the arrangement in the 1924 report in which the emphasis is placed on a series of equal spans. Moment coefficients will be given only for the case of equal spans and these will be presented with separate coefficients for live and dead load.

The next meeting of the committee has been set for June 22 and 23, 1931, in Chicago.

ADDRESS LIST OF MEMBERS AS OF OCTOBER 2, 1930

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ANALYSIS OF MOTOR VEHICLE ACCIDENTS IN CALIFORNIA

Reported by the Bureau of Research, Statistics, and Traffic Safety, Division of Motor Vehicles, California Department of Public Works

The motoring public of California started the year 1931 with a 16.08 per cent increase in motor-vehicle accidents and a resulting increase of 16.25 per cent in persons injured and a 9.44 per cent increase in deaths. These increases are computed with the January, 1930, report as a base.

New Year's day traffic was an important factor in January accidents. This is indicated by the large volume of accidents occurring on Thursdays. The same was true in January, 1930, when more accidents occurred on Wednesdays, as January 1, 1930, fell on a Wednesday.

Light conditions under which accidents occured in January, 1931, varied slightly from the average, since considerably more accidents were reported as occurring in darkness than in daylight.

The road location of accidents followed the average in January, with the most accidents at city intersections. "Between intersections" was second greatest in the number of accidents by location. It is of interest to note that 23 accidents occurred on bridges and piers, 6 of which were fatal accidents. This is an increase of over 100 per cent over January, 1930.

A total of 2,325 motorists were reported definitely at fault in January accidents. Of these 608 violated right of way, while 502 drivers came to grief because of too much speed. Nineteen of those reported as driving at excessives speed were involved in fatal accidents.

Although the total of deaths due to traffic accidents in January, 1931, showed an increase of 17 over those occurring in January, 1930, the number of pedestrians killed was 7 less. The largest numerical increases in deaths were in noncollision accidents and collisions of two motor vehicles. The number of persons killed on bicycles increased from 1 in January, 1930, to 4 in January of this year. The 1 person killed on the bicycle in 1930 was a male over 50 years of age, while all of those killed in January of this year were male children less than 20 years of age.

The number of collisions with railroad trains dropped from 61 in January, 1930, to 51 in January, 1931. The injured in this type of accident decreased by 7, but the deaths increased by 2.

Fewer accidents were reported as occurring under inclement weather conditions than in January, 1930, although the total number of accidents in January, 1931, increased more than 16 per cent over the previous year. The following table summarizes the accident statistics for the months of January, 1930, and January, 1931.

TABLE 1.—*Comparative monthly summary of motor vehicle accidents in California, January, 1930, and January, 1931*

Item	Number reported during January, 1931	Number reported during January, 1930
Accidents	2,880	2,481
Persons killed	197	180
Persons injured	3,740	3,217
Drivers involved	4,280	3,644
Pedestrians involved	984	900
Vehicles involved	4,323	3,677

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Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.
Report of the Chief of the Bureau of Public Roads, 1929.
Report of the Chief of the Bureau of Public Roads, 1930.

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DEPARTMENT CIRCULARS

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1036Y. Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio. (1927)

Report of a Survey of Transportation on the State Highways of Vermont. (1927)

Report of a Survey of Transportation on the State Highways of New Hampshire. (1927)

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio. (1928)

Report of a Survey of Transportation on the State Highways of Pennsylvania. (1928)

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.

Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.

Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.

Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

*Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

MARCH 31, 1931

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total	Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total		
		$	$				$	$				$	
Alabama	2,207.1	4,894,519.03	2,373,390.47	197.2	16.3	213.5	2,187,082.10	1,082,215.79	10.2	102.7	112.9	3,724,484.70	Alabama
Arizona	889.8	4,409,048.73	3,132,979.76	166.6	156.6	323.1	286,217.32	147,076.74	6.6	32.8	38.4	1,557,287.14	Arizona
Arkansas	1,811.9	6,998,220.81	3,257,948.10	206.6	19.5	226.1	2,565,788.61	1,187,999.92	51.0	60.2	111.2	922,151.74	Arkansas
California	1,965.1	8,267,253.82	3,523,747.83	207.4	30.8	238.2	2,865,293.89	892,191.78	76.1	10.9	87.0	2,964,988.82	California
Colorado	1,272.9	5,192,335.14	2,739,453.84	171.2	89.0	260.2	1,421,533.69	770,305.12	63.0	21.4	84.4	2,508,186.97	Colorado
Connecticut	258.2	3,293,772.72	1,346,102.89	27.8		27.9	1,149,310.11	327,514.39	11.3		11.3	309,790.28	Connecticut
Delaware	318.9	158,645.85	79,322.92	4.7		4.7	943,646.53	471,068.76	51.5		51.5	154,916.86	Delaware
Florida	640.0	4,456,392.06	2,169,110.72	108.7		108.7	879,577.36	417,791.90	35.8		35.8	2,076,433.95	Florida
Georgia	2,709.7	9,172,634.95	4,458,363.64	309.4	124.9	434.3	1,410,085.99	667,287.66	49.7	10.3	60.0	2,094,371.49	Georgia
Idaho	1,297.3	2,677,766.53	1,536,381.34	170.5	11.3	181.8	1,782,695.27	918,466.56	109.6	21.0	130.5	846,666.90	Idaho
Illinois	2,196.6	21,311,306.12	9,606,378.14	588.3	57.3	646.6	7,119,403.81	3,200,206.17	214.2	4.7	218.9	3,918,451.61	Illinois
Indiana	1,679.0	6,339,933.27	2,756,132.47	126.0		126.0	6,849,442.34	3,015,107.32	206.8		205.8	967,894.59	Indiana
Iowa	3,212.6	1,341,659.86	591,204.69	25.8	17.4	43.2	6,699,677.73	2,764,271.30	141.5	76.1	217.6	4,059.10	Iowa
Kansas	3,084.7	5,603,393.05	2,660,531.08	374.6	20.7	396.2	3,613,063.06	1,667,425.03	168.0	31.4	199.4	1,664,679.47	Kansas
Kentucky	1,555.7	5,218,771.56	2,217,725.08	178.0	81.1	259.1	3,760,288.55	1,141,496.79	82.7	55.7	138.4	648,562.02	Kentucky
Louisiana	1,417.5	7,719,132.62	3,691,621.98	258.3	17.4	276.7	1,860,133.42	768,276.05	21.4	2.6	24.0	153,801.26	Louisiana
Maine	591.3	2,310,424.19	728,331.27	48.6		48.5	2,331,384.44	1,010,338.77	63.1		63.1	1,131,743.95	Maine
Maryland	703.1	283,984.10	78,782.10		3.9	3.8	1,294,458.46	647,229.20	40.4		40.4	363,824.79	Maryland
Massachusetts	711.5	8,386,636.96	2,088,422.83	72.3		72.3	73,642.42	22,980.00	1.5		1.5	2,564,820.29	Massachusetts
Michigan	1,776.6	9,642,210.16	3,854,942.02	231.8	34.2	266.0	1,085,648.44	510,824.65	56.1	1.5	57.6	3,499,413.36	Michigan
Minnesota	4,306.4	2,003,113.07	768,662.68	22.3	82.8	115.2	6,771,663.90	2,368,708.07	64.7	175.7	230.4	673,718.49	Minnesota
Mississippi	1,808.6	1,886,702.79	939,957.86	65.8	42.2	108.0	1,949,364.59	936,100.02	64.8	41.4	106.2	4,141,500.40	Mississippi
Missouri	2,648.1	7,638,914.64	2,891,262.92	167.0	37.5	204.5	3,077,571.66	1,361,389.01	99.8	31.6	131.4	1,548,439.24	Missouri
Montana	1,671.2	8,015,684.60	4,517,527.33	630.7	55.3	686.0	3,548,832.64	1,998,450.70	239.8	96.4	335.2	2,929,482.65	Montana
Nebraska	3,894.6	6,275,495.59	2,533,097.48	245.7	70.6	316.3	2,432,165.66	1,055,330.09	33.7	123.0	156.7	2,724,635.86	Nebraska
Nevada	1,257.4	1,194,347.30	980,606.32	52.4	83.4	135.8	706,398.78	360,707.67	21.8	44.0	65.8	1,436,983.68	Nevada
New Hampshire	392.0	609,439.04	229,098.40	9.7		9.7						668,699.65	New Hampshire
New Jersey	555.2	4,446,706.15	1,346,390.37	60.5		60.5	119,849.30	59,924.65	3.5		3.6	1,431,971.74	New Jersey
New Mexico	2,020.2	4,699,076.31	3,143,300.48	172.4	48.9	221.3	2,056,891.97	1,310,579.34	76.6	69.9	145.6	260,980.16	New Mexico
New York	2,673.0	18,079,910.33	3,689,505.00	236.7		235.7	29,661,824.40	12,344,196.00	517.0	7.0	524.0	412,127.84	New York
North Carolina	1,814.0	6,507,367.12	2,640,929.15	165.8	33.5	199.3	3,351,665.94	1,644,736.53	123.0	19.2	142.2	1,727,318.69	North Carolina
North Dakota	4,550.5	2,087,233.17	1,084,437.89	270.4	133.2	403.6	1,953,879.32	976,638.65	234.2	292.3	626.5	2,072,329.51	North Dakota
Ohio	2,653.2	10,976,392.49	3,346,196.42	147.4	13.2	160.6	3,198,560.83	996,805.81	42.8	15.9	58.7	3,871,247.80	Ohio
Oklahoma	1,979.3	5,018,684.68	2,383,202.70	191.8	86.7	258.6	4,190,287.95	2,294,368.63	163.0	64.8	227.6	194,978.35	Oklahoma
Oregon	1,272.7	6,619,503.68	3,630,486.02	182.3	80.6	272.9	830,691.98	328,200.07	49.2	9.9	69.1	1,211,161.70	Oregon
Pennsylvania	2,658.1	7,304,923.48	2,600,346.48	47.0		47.0	404,741.83	69,187.89	2.9		2.8	5,183,597.04	Pennsylvania
Rhode Island	209.6	1,912,612.98	761,194.76	31.6		31.6	374,296.87	158,994.01	6.3		6.3	398,028.85	Rhode Island
South Carolina	1,798.8	7,276,114.96	3,204,866.40	97.4	192.8	290.2	237,006.12	118,502.56		11.2	11.2	122,309.45	South Carolina
South Dakota	3,786.8	3,411,948.28	1,970,740.20	305.0	66.7	371.7	1,241,982.44	672,141.64	125.6	64.5	190.1	1,656,937.62	South Dakota
Tennessee	1,448.6	3,244,342.67	1,620,847.47	142.0		142.0	1,885,269.67	924,118.77	82.3	13.4	95.7	2,046,515.96	Tennessee
Texas	7,013.4	14,244,602.90	6,197,945.07	671.9	163.7	835.6	7,363,002.48	3,502,890.21	226.0	202.1	427.1	6,271,883.36	Texas
Utah	985.2	1,838,027.51	1,153,291.04	127.7	49.2	176.9	896,765.19	367,461.84	35.2	54.7	89.8	1,108,727.62	Utah
Vermont	303.6	304,277.34	133,620.98	3.2		3.2	960,743.63	386,397.22	23.0		23.0	172,183.63	Vermont
Virginia	1,633.6	4,849,686.85	2,315,051.16	210.7	21.8	232.3	1,059,073.03	336,870.01	34.1	18.0	52.1	1,171,253.49	Virginia
Washington	1,011.1	3,841,967.98	1,783,137.61	127.6	7.7	135.3	1,294,418.39	542,428.75	40.8	12.1	52.9	1,638,767.92	Washington
West Virginia	765.5	4,594,359.69	1,736,454.31	103.6	12.2	116.8	1,464,879.70	644,194.41	36.6	.3	36.9	447,386.80	West Virginia
Wisconsin	2,476.3	3,482,772.06	1,492,374.56	81.8	13.0	94.8	1,284,689.31	572,489.81	50.3	11.7	62.0	2,234,059.16	Wisconsin
Wyoming	1,708.9	3,100,149.80	2,008,346.01	197.5	146.0	343.6	611,757.68	344,850.15	63.8	50.3	104.1	891,809.11	Wyoming
Hawaii	46.3	790,777.25	361,304.04	26.8		26.6	623,763.91	205,849.56	13.5		13.5	1,631,740.97	Hawaii
TOTALS	89,658.6	260,713,862.01	114,443,942.17	8,277.1	2,111.3	10,388.4	132,801,079.38	58,459,564.86	3,909.8	1,859.4	6,769.2	81,016,163.92	TOTALS

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

U. S. GOVERNMENT PRINTING OFFICE 1931

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 12, NO. 3 ▽ MAY, 1931

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A BITUMINOUS SURFACE TREATMENT OF A LIME ROCK ROAD IN FLORIDA

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

VOL. 12, NO. 3 MAY, 1931 G. P. St. CLAIR, Editor

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AN INVESTIGATION OF OIL-TREATED EARTH ROADS IN MISSOURI

Reported by F. V. REAGEL, Engineer of Materials, Missouri State Highway Department, HENRY AARON, Assistant Highway Engineer, U. S. Bureau of Public Roads, and W. I. WATKINS, Assistant Soil Surveyor, U. S. Bureau of Chemistry and Soils

THE best efforts of almost every State highway organization in the past have been devoted to the design and the construction of high-type roads to serve the demands of ever-increasing traffic on the more important routes of the respective States. The recognition of these demands for high-type roads caused the design of feeder roads connecting with the main highways to receive but little attention. In Missouri, as in several other States, the primary road system is approaching completion and the demand for the construction of secondary, light-traffic roads is rapidly increasing.

In anticipation of this demand for low-cost roads, the Missouri State Highway Department, in cooperation with the United States Bureau of Public Roads and the United States Bureau of Chemistry and Soils, has been investigating the practicability of oil-treated roads, with or without subsequent stages of improvement, for use in those sections, comprising approximately one-third of the total area of the State, where no surfacing material of any kind is locally available. The collecting and organization of the data and observations on application of the oil and the condition surveys on the sections studied and included in this report were carried out by R. C. Schappler and C. M. Lancaster, of the division of geology and soils of the Missouri State Highway Department.

The roads included in this investigation were oiled during the months of May to November, 1928, and the condition of the surface as disclosed by condition surveys was recorded at various times up to May, 1929. The first inspection of the surface condition was made about two and one-half months after the oil was applied. A fall condition survey was made during the months of September to December, 1928, about one and one-half months after the first inspection. On several of the sections treated in the fall, the first inspection after treatment served also as the fall condition survey. The final detailed condition survey included in this report was made the following spring during the period from March to May, 1929.

During the following two years the department's program included a considerable mileage of treating and retreating earth roads with road oil. This program provided an opportunity for demonstrating and establishing the indications and conclusions developed in this study with increasing confidence in the design and methods of road oil application. A supplementary condition survey was made in the spring of 1931.

The investigation has disclosed that oils may be used satisfactorily for the treatment of earth surfaces to provide temporary all-weather roads if certain fundamental factors are recognized. The most important factors so far discovered are: (1) Physical characteristics of the soil; (2) drainage, both surface and subsurface; (3) condition of surface immediately prior to application of oil; (4) rate of penetration of oil; (5) type of traffic; and (6) methods of maintenance.

CHARACTER OF ROAD SECTIONS INVESTIGATED

Prior to 1928, in those sections of Missouri which are deficient in local road-building materials, about 200 miles of road had been constructed, consisting of 9-foot concrete pavement on one-half of the roadway and graded earth on the other half. This type of improvement was utilized in order to hasten the connection of several communities by all-weather traffic lanes. The resulting roads furnished the service expected but with a serious dust hazard on some soils during dry periods and an expensive mud nuisance in wet weather. The graded portions of these roads were then oiled in an effort to alleviate these conditions. In addition, approximately 100 miles of full-width graded earth roads have been oiled. Twenty-nine sections of road, varying in length from 2 to 21 miles, with a total length of 224 miles, were selected for special study. The locations of these sections and their approximate lengths are given in Table 1.

The topography of the area studied varies from level to rolling. The extent of the rolling character controls the relative amount of cut and fill on the sections. This feature in turn governs the amount of variation in soils in cuts due to uncovering of different soil layers at varying depths, as well as the resultant character of soil occurring in the mixture from which the fills were made.

The roads selected for oil treatment were constructed according to standard design. Drainage as a whole had been taken care of. Occasional sections which, because of improper drainage, had not been maintained to the standards of the original construction, could be readily noted by observation.

Traffic on these highways is mixed and was classed as mutilative and nonmutilative. Mutilative traffic included tractors, threshing outfits, and other machinery, the wheels of which were equipped with lugs or cleats, and also horses hoofs' and wagons having narrow-tired wheels.

TABLE 1.—*Location and mileage of sections of oiled road given special study*

Section No.	Location	Approximate length
		Miles
1	State route 6, DeKalb County, Bayfield to Oak	8
2	State route 31, DeKalb County, King City to Oak	11
3	U. S. route 69, Daviess County, Cameron north	5
4	State route 6, Daviess County, Winston to Altamont	7
5	State route 6, Daviess County, Altamont to Gallatin	8
6	U. S. route 69, Harrison County, Bethany to Iowa line	21
7	State route 59, Platte County, Parkville north	8
8	State routes 1 and 9, Atchison County, Tarkio to Rockport	10
9	State route 1, Atchison County, Fairfax north	9
10	U. S. route 65, Saline County, Marshall to Salt Fork Creek	6
11	U. S. route 65, Saline County, Salt Fork Creek to Grand Pass	9
12	U. S. route 65, Saline County, Grand Pass west	2
13	U. S. route 24, Carroll County, near DeWitt	3
14	U. S. route 63, Randolph County, Jacksonville to Moberly	10
15	U. S. route 24, Randolph County, Huntsville west	9
16	State route 10, Ray County, Richmond to Hardin	8
17	U. S. route 71, Platte County	
18	U. S. route 71, Platte County	
19	U. S. route 71, Platte County	12
20	U. S. route 71, Platte County	
21	U. S. route 71, Platte County	
22	U. S. route 63, Macon County. Extends approximately 4 miles north from Axtell	4
23	U. S. route 63, Macon and Adair Counties	17
24	U. S. route 63, Adair County	3
25	U. S. route 65, Grundy and Livingston Counties, between Chillicothe and Trenton	8
26	State route 6, DeKalb County, Maysville to Oak	7
27	U. S. route 63, Boone County, Hinton to Clark	13
28	State route 13, Lafayette County, Higginsville to Lexington	10
29	State route 10, Ray County, Excelsior Springs to Richmond	16
	Total mileage	224

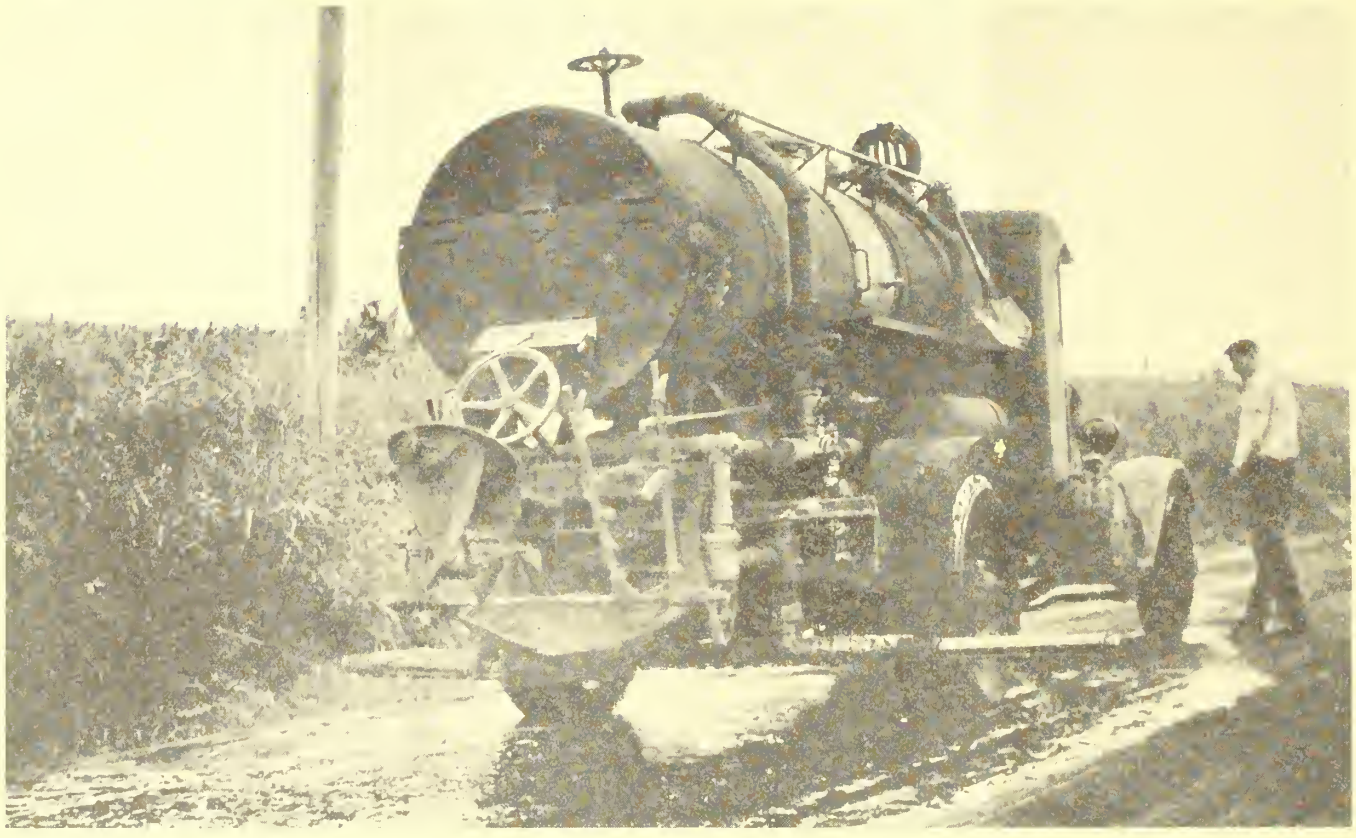


FIGURE 1.—EIGHT HUNDRED GALLON PRESSURE DISTRIBUTOR READY TO BEGIN APPLICATION. OVERLAPPING OCCURRED IN APPLYING SUCCESSIVE LOADS

TYPE OF OIL USED

On the greater portion of the mileage a single type of oil meeting the State highway specifications, and similar to oil No. 1 (Table 3) was used. On certain special sections the type of oil was made a variable, and on these sections other types of oil and combinations were used. Specifications for these oils can be found in Table 2, and typical analyses in Table 3. A brief description of the oils follows this latter table.

TABLE 2.—*Specifications for road oils*

The oil shall be homogeneous, free from water, and shall not foam when heated to 120° C. (248° F.), and shall meet the following requirements:

Item	Specification A	Specification B
1. Specific gravity 15.5°/15.5° C., not less than	0.910	1.00
2. Specific viscosity (Engler) at 60° C	10 to 20	10 to 20
3. Flash point, °C., not less than	80	90
4. Per cent loss at 163° C., 50 grams, 5 hours, not more than		6
5. Total bitumen (soluble in CS_2), per cent, not less than	99.5	99.5
6. When the specific gravity of the oil at 15.5° C. is less than 0.970, the percentage of bitumen insoluble in 86° Baumé paraffin naphtha shall not exceed	8.0	
7. Per cent bitumen insoluble in 86° Baumé naphtha, not less than		9.0
8. Residue of 100 penetration (100 grams, 5 seconds, 25° C.) per cent	50–65	50–65
9. Ductility of residue, centimeters (when 50 grams of the oil are evaporated at 250° to 260° C. until the residue has a penetration at 25° C., 100 grams, 5 seconds, of 90 to 100), not less than		50

CONSTRUCTION METHODS OUTLINED

A contract was let for the oiling under the supervision of the State bureau of maintenance, and a schedule was arranged to take care of the order in which the sections would be treated. The various local maintenance forces were required to have their sections ready for oiling according to this schedule. This work was accompanied by the normal amount of confusion and delay due to unforeseen weather conditions or failure of equipment. In general, the preparation consisted of intensive blading and dragging to provide a smooth and uniform surface for the oil. Ditches were recut wherever necessary. Considerable variation was noted in the thoroughness with which this work was carried out. The variations are explained by the fact that the individual sections were prepared by different, and, to some extent, independent forces.

The oil was discharged from pressure distributors as shown in Figure 1. The temperature of the oil as applied varied from 120° F. to 200° F. The rule on

TABLE 3.—*Typical analyses of oils used*

Oil No.	Specific gravity, 15.5°/15.5° C.	Bitumen soluble in CS_2	Specific viscosity, Engler at 60° C.	Flash point	Residue of 100 penetration	Bitumen insoluble in 86° Baumé naphtha	Ductility of specified residue [1]	Loss at 163° C., 50 grams, 5 hours
		Per cent		*° C.*	*Per cent*	*Per cent*	*Cms.*	*Per cent*
[2] 1	.937	99.94	11.31	207	58.72	1.4	3.7	1.24
[2] 2	.949	99.77	15.00	210	62.50	2.7	150	(3)
[4] 3	1.011	99.78	10.82	180	62.34	11.82	150	4.09
[5] 4	.9415	99.96	14.97	158	58.92	2.47	150	(3)
[6] 5	.966	99.97	14.14	105	60.00	8.04	150	(3)

[1] Ductility of specified residue, in centimeters, when 50 grams of the oil are evaporated at 250° to 260° C. until the residue has a penetration, at 25° C., 100 grams 5 seconds, of 90 to 100.
[2] Topped residual from eastern Kansas crude, with a high paraffin base.
[3] Less than 5.00.
[4] Residual from cracking process from general midcontinent crude.
[5] Topped residual from southern Oklahoma crude, having a semiasphaltic base.
[6] Low pressure still product from Gulf coastal crude, having an asphaltic base.

most sections was to obtain an oil temperature of at least 140° F. The permissible moisture in the earth was limited to the amount present when ruts did not develop under the weight of the distributor. Oil was applied at the rate of one-half gallon per square yard for the first application, followed as soon as the oil had been absorbed so that it would not be picked up by traffic by a second application of one-fourth gallon per square yard. The quantity per square yard was controlled by computing from the measured gallonage of the truck-load the length of road to be covered by the load. This distance was chained off and flag-staked for the truck-driver to reach at uniform speed while distributing the truck-load. In some instances traffic was allowed to use half of the roadway while oiling was in progress; in others, the roads were barricaded and closed to traffic until the second application had been absorbed. All sections noted as re-treated received a third application of one-fourth gallon per square yard.

In a few cases some material, generally river sand, was applied in a thin layer to the surface.

SECTIONS MAINTAINED CHIEFLY BY BLADING

Fifteen of the 29 sections investigated were bladed to some extent. On some sections the blading consisted of merely spreading a thin layer of earth from the shoulder over the treated surface, without permitting the blade to come in contact with the surface. On other sections attempts were made to eliminate defects by shaving the treated surface slightly and filling the defects with the material thus obtained.

OBSERVATIONS MADE BEFORE, DURING, AND AFTER TREATMENT

For each section studied the investigation involved the following activities:

1. Obtaining information relative to conditions existing and preparations made prior to the application of the oil.
2. Recording pertinent information at the time of treatment.
3. Making periodic surface condition surveys subsequent to treatment.
4. Making special studies of the subgrade soil.

The information relative to conditions existing and preparations made prior to the application of the oil included a record of the weather conditions existing prior to oiling, the soil types, the road profile, the drainage conditions, the type of equipment used, the surface processing, the length of time the surface was maintained at a finished grade prior to the application of oil, the final preparation of the grade, the surface contour, the state of consolidation of the soil, and the amount of moisture present in the soil immediately before the application of the oil.

The notes taken during construction included a record of prevailing weather conditions during the oiling operations; the type of oil used; the oil temperatures; the quantity of oil applied per square yard; the average depth of oil penetration 24, 48, and 72 hours after application; the character of penetration with respect to whether it was selective, uniform, or nonuniform; the length of time required for the oil to penetrate to an extent preventing the mat from peeling under traffic; and the method of traffic control utilized during the oiling operation.

Data regarding depth, rate, and character of penetration were obtained by driving a tube into the surface at desired intervals of time and examining the cylinder of treated soil so obtained.

The surveys subsequent to treatment included the making of general condition surveys, all distances being checked by speedometer readings; a record of road surface conditions; and an estimate of the probable causes of surface defects, such as mutilative traffic, insufficient oil, inadequate drainage, etc. The results of these surface condition surveys were then correlated with the information obtained prior to and during oil treatment.

In these surveys the road surfaces furnished by the bituminous treatments were arbitrarily classified with respect to their condition in the terms which are given and described below. Examples of each classification are given in Figure 2.

Mat intact, surface smooth.—This term indicates excellent bituminous mat surfaces. The oil-earth mat, or penetration layer, is pliant and possesses a tough, leathery texture. This type of mat occurs when the bituminous material penetrates the soil in such a manner as to bind the soil particles properly and seal the road surface, thus producing a compact mat which resists the effects of weather and traffic.

Surface rutted.—This term indicates the presence of indentations or grooves in the road surface. Rutting occurs when the subgrade is softened because of the presence of moisture. It is generally preceded by a decrease in the stability of the supporting soil, which can usually be traced to moisture furnished by rains or melting snow occurring on top of the road, by seepage or capillarity beneath, or by a combination of both.

Mat crumbled.—This term indicates a crumbling of the oil-earth mat caused either by an insufficient amount of oil or by a deadening of the oil fraction. This term is not used when failure can in any way be traced to a decrease in the stability of the supporting soil. Thus, crumbling is entirely a surface defect and indicates that the applied bituminous material either has not properly penetrated the soil or does not possess cohesion in amount sufficient to bind the soil particles into a compact mat.

Mat raveled.—This term indicates a progressive breaking up, but not a crumbling of the mat. It generally occurs after periods of alternate freezing and thawing and suggests vertical displacement of the mat. This vertical displacement may be produced by expansion of the subgrade due either to increase in moisture content or to freezing followed by shrinkage of the subgrade caused by reduction in moisture content or thaw, thus leaving the mat suspended without support. Nonuniform subgrade soil is especially productive of raveling.

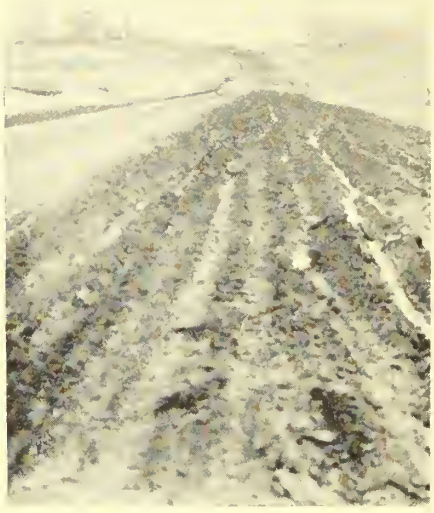
Mats on soils consisting of thin laminated layers of both silt and very fine sand are likely to bulge as much as one-fourth to one-half of an inch between wheel tracks 2 to 4 inches apart. It is indicated that in the silt soils the laminations may be the result of some textural assortment caused by rain, traffic, wind, or running water or by difference in degree of compaction, oil absorbed, etc. The bulging causes the different textural layers to be separated from each other.

Surface pitted.—This term indicates the presence of abrupt surface cavities. These cavities usually extend appreciably below the depth of soil penetrated with oil and do not include indentations or depressions not causing a "breaking through" of the oil-earth mat.

Surface checked.—This term indicates the occurrence of shrinkage of the surface soil disclosed by "alligator

MAT INTACT, SURFACE SMOOTH

SURFACE DEEPLY RUTTED



SURFACE RUTTED

MAT CRUMBLED

SURFACE PITTED AND RAVELED

SURFACE PITTED AND CHECKED

SURFACE CHECKED

SURFACE SCALED

FIGURE 2.—EXAMPLES OF SURFACE CONDITIONS NOTED IN SURVEYS

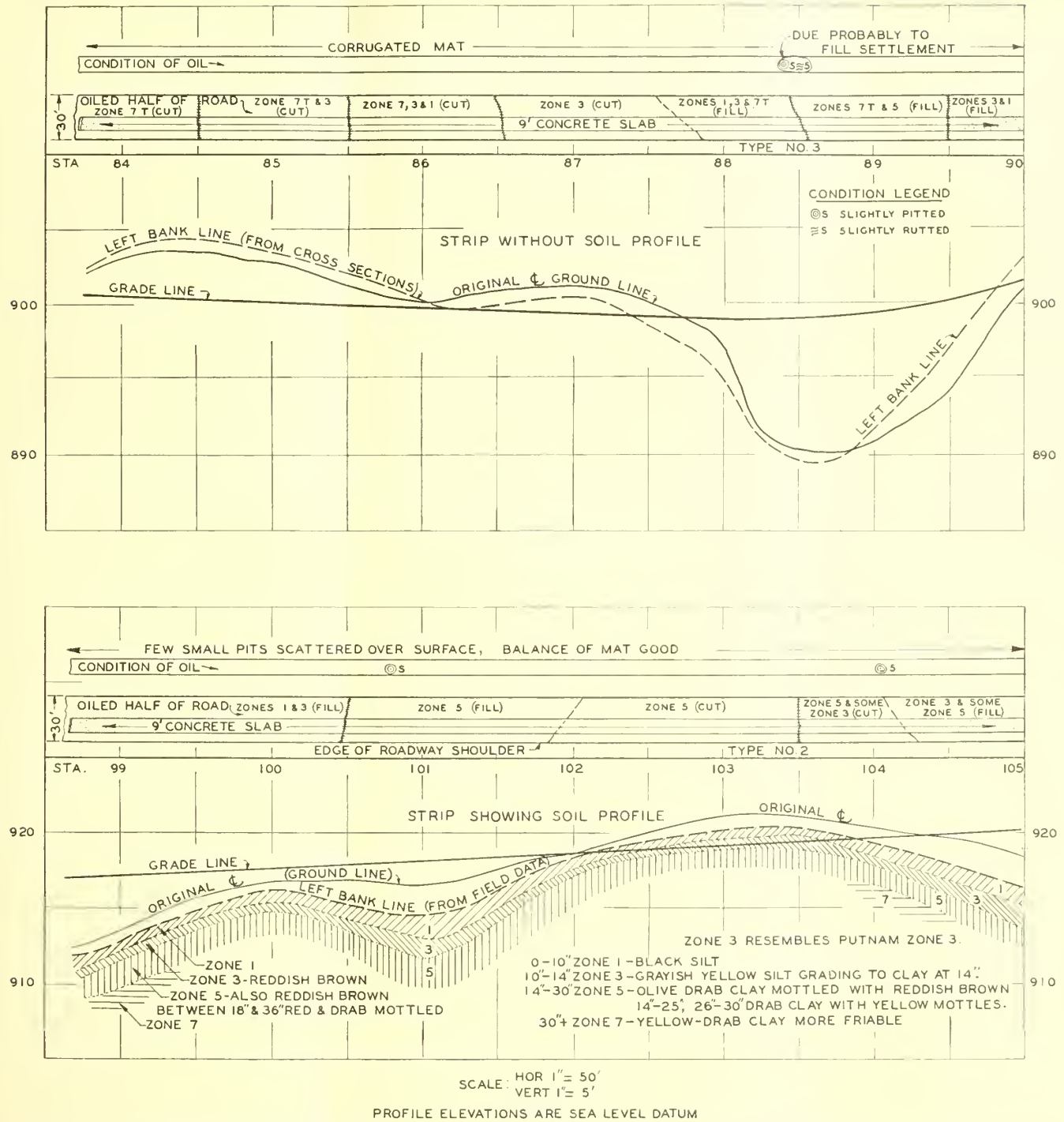

CORRUGATED MAT
DUE PROBABLY TO FILL SETTLEMENT
CONDITION OF OIL
OILED HALF OF ROAD
ZONE 7 T (CUT)
ZONE 7 T & 3 (CUT)
ZONE 7, 3 & 1 (CUT)
ZONE 3 (CUT)
ZONES 1, 3 & 7 T (FILL)
ZONES 7 T & 5 (FILL)
ZONES 3 & 1 (FILL)
30'
9' CONCRETE SLAB
TYPE NO. 3
STA 84 85 86 87 88 89 90
CONDITION LEGEND
S SLIGHTLY PITTED
S SLIGHTLY RUTTED
STRIP WITHOUT SOIL PROFILE
LEFT BANK LINE (FROM CROSS SECTIONS)
GRADE LINE
ORIGINAL ℄ GROUND LINE
LEFT BANK LINE
900
890
FEW SMALL PITS SCATTERED OVER SURFACE, BALANCE OF MAT GOOD
CONDITION OF OIL
OILED HALF OF ROAD
ZONES 1 & 3 (FILL)
ZONE 5 (FILL)
ZONE 5 (CUT)
ZONE 5 & SOME ZONE 3 (CUT)
ZONE 3 & SOME ZONE 5 (FILL)
9' CONCRETE SLAB
EDGE OF ROADWAY SHOULDER
TYPE NO. 2
STA. 99 100 101 102 103 104 105
STRIP SHOWING SOIL PROFILE
ORIGINAL ℄
GRADE LINE
ORIGINAL ℄ (GROUND LINE)
LEFT BANK LINE (FROM FIELD DATA)
ZONE 1
ZONE 3-REDDISH BROWN
ZONE 5-ALSO REDDISH BROWN
BETWEEN 18" & 36" RED & DRAB MOTTLED
ZONE 7
ZONE 3 RESEMBLES PUTNAM ZONE 3.
0-10" ZONE 1-BLACK SILT
10"-14" ZONE 3-GRAYISH YELLOW SILT GRADING TO CLAY AT 14".
14"-30" ZONE 5-OLIVE DRAB CLAY MOTTLED WITH REDDISH BROWN
14"-25", 26"-30" DRAB CLAY WITH YELLOW MOTTLES.
30"+ ZONE 7-YELLOW-DRAB CLAY MORE FRIABLE
920
910
SCALE: HOR 1" = 50'
VERT 1" = 5'
PROFILE ELEVATIONS ARE SEA LEVEL DATUM

FIGURE 3.—MAP SHOWING PLAN OF ROAD, PROFILES, AND RESULTS OF CONDITION AND SOIL SURVEYS FOR U. S. ROUTE 63, MACON COUNTY, MO., 1½ MILES SOUTH OF LA PLATA

hide" cracking which separates the oil-earth mat into irregularly shaped blocks.

Surface scaled.—This term signifies the scaling off of the oil-earth mat in thin layers. Scaling is caused generally by a laminated or plated structure existing in the oil-earth mat.

SOIL SURVEYS MADE ON SEVERAL ROADS

The special studies of the subgrade soils included the surveying of certain roads with respect to the subgrade soil types and their soil zones or layers and the testing of representative samples from the various layers in the subgrade laboratory.

The subgrade survey consisted of examining the subgrade at intervals close enough (about 100 feet generally) to determine any change in the subgrade soils and examining the soils in the adjoining fields to determine the soil profiles of the soil types encountered. This information was plotted as shown in Figure 3. Included in this record are the road plan, the profile of one of the banks, the original center line grade, and the present center line grade. A soil profile such as shown in the lower strip, Figure 3, was taken only at such intervals as were necessary to disclose the arrangement of soil layers in a given soil type. In addition, the

TABLE 4.—*Record of observations on 29 oil-treated sections*

Section No.	Oil No.	General profile	Surface drainage rating [1]	Width of untreated earth shoulder	Texture classification, principal soils	Surface condition just prior to first application	Subgrade moisture, depth, 0 to 1 inch	Prevailing weather conditions during application	Air temperature range	Average oil temperature	Penetration		
											Average depth	Days required	Nature
			Per cent	*Feet*					° *F.*	° *F.*	*Inches*		
2	1	Rolling	75–100	6	Clay and silty clay	Firm	Moist	Fair	75–90	140	0.50	14–28	Uniform.
1	3	do	50–75	12	Silty clay	Retreatment, bituminous residue.		do	70–95	130		2–4	Do.
8	1	do	50–75	0	Silt	Firm	Moist	Occasional showers.	70–95	150	.40	4–7	Do.
15	1	do	75–100	12	Silt loam and silty clay.	do	do	Fair	55–85	145	.50	4–7	Do.
26	1	do	75–100	12	Clay and silty clay	Firm, some dust	do	Occasional showers.	80–95	135	.60	7–14	Do.
28	4	do	50–75	12	Silt loam	do	do	Cool and fair	45–70	190	.60	14–28	Do.
29	4	do	50–75	6	No record	No record	No record	do	45–70	(2)	(2)	14–28	No record.
4	1	do	50–75	6	Clay	Firm, some dust	Moist	Occasional showers.	60–90	200	.45	4–7	Uniform.
5	1	do	50–75	6	Silt loam and clay	do	do	do	60–90	190	.45	4–7	Do.
6	1	do	50–75	6	Silty clay and clay	Soft and loose	Wet	Frequent rains	65–85	150	.40	14–28	Do.
7	1	do	50–75	12	Silt loam	Firm, some dust	Moist	Occasional showers.	75–90	150	.70	2–4	Nonuniform.
9	1	Level	50–75	0	Silty clay	do	do	do	70–90	150	.50	4–7	Do.
14	1	do	50–75	6	Silt loam	do	Dry	Fair	60–80	160	.60	2–4	Do.
22	3	do	50–75	6	do	do	do	do	70–85	150	.85	1–2	Nonuniform and variable in color.
23	1	Rolling	50–75	6	Silt loam and silty clay.	do	do	do	65–85	150	.60	4–7	Nonuniform.
3	3	Level	25–50	6	Silt loam	Loose and dusty	do	do	60–90	200	.75	1–2	Do.
10	3	Rolling	25–50	6	do	Hard and dusty	do	Occasional showers.	80–90	160	.70	1–2	Nonuniform and selective.
11	1	Level	25–50	6	do	do	do	do	80–95	160	.65	2–4	Do.
12	3	Rolling	25–50	6	do	Very dusty	do	do	80–95	150	.70	1–2	Nonuniform.
13	3	Level	0–25	12	Silty clay	Firm	do	Fair	80–95	150	.55	4–7	Uniform.
16	3	do	25–50	12	do	Hard, partly crusted.	do	do	65–80	130	.55	4–7	Selective and variable in color.
24	2	Rolling	25–50	6	Silt loam	Hard and dusty	do	Cool and cloudy.	55–70	180	.50	2–4	Nonuniform and selective.
25	1	do	25–50	6	Silty clay and clay	Hard, partly crusted.	do	Fair	60–80	160	.40	2–4	Selective.
27	1	do	50–75	6	Silt loam and silty clay.	Very dusty	do	do	75–90	150	.50	2–4	Nonuniform.
17	1	do	0–25	12	Silt loam	Hard, partly crusted	do	Occasional showers.	70–80	146	.70	1–2	Nonuniform and selective.
18	1, 3, 5	do	25–50	12	do	do	do	do	65–80	170	.60	1–2	Variable in color and selective.
19	1, 3	Level	25–50	12	Silty clay	do	do	do	80–95	135	.60	1–2	Do.
20	3	do	25–50	12	do	do	do	do	80–95	140	.50	1–2	Do.
21	3, 5	do	25–50	12	do	do	do	do	80–95	140	.40	1–2	Do.

[1] See text, p. 57.

[2] No record.

drawings include the information furnished by the condition surveys made of the road surface.

The soil comprising the roadway is indicated by a zone number which corresponds to the same number and soil layer in the soil profile. When the subgrade consists of mixed materials from several zones, the zone numbers are shown in the order in which the materials predominate in the mixture; thus "zones 1, 3, and 5" indicates that soil material of zone 1 seems to dominate zone 3 material and that zone 3 material seems to dominate zone 5 material in amount. The numerals give only a general idea of the materials, as fills are composed of a heterogeneous instead of a homogeneous mixture of materials from the various zones. The soil profile of Figure 3 is taken along the left bank line while the soil data shown on the corresponding plan are based on an examination of the roadway.

The original center line grade was determined from the amounts of both cut and fill indicated by the original construction cross sections sheets of the roads studied. Thus the original center line grade does not necessarily mean the original ground line, except on newly located roads. In other cases the original ground line can be estimated only approximately from the present bank line profiles.

The soil profiles were determined by an examination of the back slopes and borings. The soil material in the subgrade was determined by using a pick and examining the soil to a depth of 3 or 4 inches. The soil in the oil mat was considered to be similar in character to that supporting the oil mat.

The soil in the subgrade may differ from that which would be expected from the bank profile. This may be due to several factors, as follows: (*a*) The material may be fill over an old roadbed; (*b*) material from ditches or back slopes obtained from a zone above or below the zone on the level with the subgrade may have been used in finishing the road; and (*c*) the original materials may have been rearranged by blading.

DATA PRESENTED IN TABULAR FORM

For convenience of analysis the records of the observations on the 29 sections studied are given in Table 4. The sections are listed in the order of their performance as shown by the first complete condition survey.

Some of the variables studied were not determined with great accuracy. For example, variations in moisture content, because of gradations in soil type and differences in elapsed time between preparation of surface and application of oil, could not be determined readily, hence descriptive terms depicting average conditions were used. Such terms could be easily analyzed and were sufficiently accurate for the purpose at hand. Data listed as average are available in detail on specific sections, in most cases, but are more susceptible of analysis after summation.

SOILS IN TWO ROAD SECTIONS GIVEN SPECIAL STUDY

The soils encountered in the special investigation of U. S. Highway No. 65, Waverly to Marshall, stations

TABLE 4.—*Record of observations on 29 oil-treated sections*—Continued

Section No.	Oil No.	Cover material	Character of traffic after treatment	Surface after treatment		Surface condition in late autumn	Surface condition in early spring	Manipulation after treatment	Remarks
				Months after	Character				
2	1	None	Nonmutilative	2	Live and ductile	Excellent	Good	None	Oil applied to surface immediately after final blading.
1	3	do	do	2	Hard but malleable	Good	Fair	do	
8	1	do	do	2	Live and ductile	do	do	do	Maintained very carefully; no blading. Defects repaired with premixed oil and earth.
15	1	do	Mutilative	1	do	do	do	do	Oil applied immediately after final blading.
26	1	do	do	3	do	do	do	do	Do.
28	4	do	do	1	do	do	do	do	Do.
29	4	do	Nonmutilative	1	do	do	do	do	Do.
4	1	do	Mutilative	5	do	Fair	do	Bladed outer edge.	Portions of surface very damp at time of application.
5	1	do	do	5	do	do	do	do	
6	1	do	Nonmutilative	3	do	do	do	None	Portions of surface very muddy at time of application.
7	1	1 inch sand.	Mutilative	2	do	Fair, retreated	Good	Dragged	Oil applied 12 to 24 hours after final blading.
9	1	None	Nonmutilative	2	do	Fair	Fair	do	Very carefully maintained.
14	1	do	Mutilative	4	Hard, some pitting	do	Poor	Bladed	Abrupt differences in surface consolidation; numerous dust pockets.
22	3	do	do	3	Dead and dusting	Poor, retreated	Fair	Dragged	Do.
23	1	do	do	3	do	do	do	do	Do.
3	3	do	do	5	do	do	do	Bladed	Insufficient oil to bind soil particles to depth penetrated.
10	3	do	do	3	do	do	Poor	do	Do.
11	1	do	do	3	do	do	do	do	Do.
12	3	do	do	3	do	do	do	do	Do.
13	3	2-inch sand.	do	1	Live and ductile	Poor	do	do	Unstable support due to subsurface drainage conditions.
16	3	3-inch sand.	do	1	Dead and dusting	do	do	do	Traffic used selective lanes because of uneven and rutted surface.
24	2	None	do	3	do	Poor, retreated	do	Dragged	
25	1	do	do	4	do	do	do	Bladed	
27	1	do	Insufficient	1	do	Poor	do	do	Oil deadened rapidly because of lack of traffic during 2 months after treatment.
17	1	do	Mutilative	2	Dead and pitting	Failed	Failed	do	Surface was prepared for treatment 1 month before oil was applied, and during this iterim no blading was done. Final preparation consisted of smoothing the surface by means of a drag and maintainer. As a result considerable portions of the surface were hard and crusted, while other portions were composed of unconsolidated material.
18	1, 3, 5	do	do	2	Malleable, some pitting	do	do	do	
19	1, 3	do	do	2	Dead and pitting	do	do	do	
20	3	do	do	2	do	do	do	do	
21	3, 5	do	do	2	Malleable, but pitting	do	do	do	

0 to 980 (sections 10 to 12 in Tables 1 and 4), and U. S. Highway No. 63, Axtell to La Plata, stations 34 to 840 (sections 22 and 23 in Tables 1 and 4), belong principally to the Marshall and Putnam series as designated by the United States Bureau of Chemistry and Soils. Lesser areas of Knox silt loam, Shelby loam, an uncorrelated type designated as type No. 2 on U. S. Highway No. 63 and an uncorrelated type designated as type No. 3 on U. S. Highway No. 65, were mapped. A brief description of these soils is given here.

U. S. Highway No. 63, Axtell to La Plata.—The soil profile of the Putnam silt loam which occupies the flat prairie regions of north central and northeastern Missouri is composed of five distinct soil zones or layers. The first layer, zone 1, is a grayish brown, fine granular or crumb-structured friable silt loam. The second layer, zone 3, is a gray, laminated platy or flaky silt loam. The third layer, zone 5, is a brown, heavy, tough plastic clay which breaks into small rectangular or square particles having a smooth outer coating and are hard when dry. These particles become larger and less stable with depth. The fourth layer, zone 7, is a bluish gray structureless, heavy, sticky, plastic clay containing varying amounts of yellow mottles. Small quantities of sand may occur in spots through the lower part of the fourth layer. The fifth layer, zone 9, is a structureless yellow or yellow-and-gray-splotched, slightly oxidized, calcareous, glacial clay, or clay loam containing varying percentages of sands and silts.

Bordering the Putnam silt loam in the timbered areas, the uncorrelated type No. 2 was mapped. This type possesses the same number of soil zones as the Putnam silt loam and differs mainly in the second layer, zone 3. In place of the gray laminated silt layer of the Putnam silt loam, a yellow to yellowish-gray fine granular silty clay loam occurs.

A soil type derived from glacial drift and developed under forested conditions was mapped on the rolling topography of this highway. This type closely resembles the Shelby loam of northern Missouri. The first layer, zone 1, is a brown friable loam or silt loam of fine granular structure. The second layer, zone 3, is a brown to grayish-brown granular loam or clay loam. The third layer, zone 5, is a yellowish brown, friable, subangular-structured clay loam or clay. The fourth layer, zone 7, is a yellow-and-gray-mottled structureless glacial clay or clay loam containing varying percentagesof sand.

U. S. Highway No. 65, Waverly to Marshall.—The Marshall soils (silt loam and deep and shallow phases) of the prairie regions of central Missouri were encountered on practically the entire length of the portion of this highway studied. The first layer, zone 1, of the silt loam is a dark brown, friable, finely granular or crumb-structured silt loam. The second layer, zone 3, is a brown silt loam of slightly coarser structure and heavier texture than the first layer. The third layer, zone 5, is a brownish-yellow silt loam of no definite structure. The fourth layer, zone 7, is a gray silt loam with yellow splotches. The fifth layer, zone 9, is a gray silt with iron concretions developed along decayed roots. The deep and shallow phases of this type varied only slightly from the above description.

The Knox silt loam occupies the more rolling topography lying between the Marshall soils and the Missouri River and is very similar to the Marshall soils with the exception that it is lighter colored.

The uncorrelated type No. 3 is a prairie soil derived from loessial material. The first layer, zone 1, is a dark brown silty clay loam of granular structure. The second layer, zone 3, is a bluish drab, sticky, plastic, granular, structured clay. In the flat areas having inferior drainage another layer, zone 2, a black, slightly sticky clay of crumb structure, is found between the first and second layers. The third layer, zone 5, is a bluish-gray-and-yellow-mottled, sticky, plastic clay or silty clay loam. The fourth layer, zone 7, is a bluish gray, mottled with yellow, structureless clay of high silt content.

The reaction of the individual soil zones with oil as shown by the condition survey is tabulated in Table 5, together with the laboratory test results of representative samples of soil from each of the soil zones encountered. The soil zones are assigned to certain soil groups according to their physical characteristics as disclosed by laboratory tests.

BENEFITS DERIVED FROM OIL TREATMENT DEPEND ON CHARACTER OF SUBGRADE SOIL

No definite major effect of soil type is developed from a study of Table 4 but when the data in this table are combined with those obtained from the more detailed examination and mapping of soils on U. S. Highways 63 and 65, as given in Table 5, the difference in reaction of the various layers of the different soil types with oil is clearly indicated. In general, all classes of soil were benefited by the oil treatment, but the degree of success obtained varied with the physical characteristics of the subgrade soil. This fact indicates that the requisites of the various soil layers and soil types are different.

Uniformly good results, other conditions being favorable, were obtained by treating the cohesive type of soil (clay) regardless of the character of the base of the oil, whereas better results were obtained in the case of the noncohesive type of soil (silts) with the use of oil having greater viscosity and a more ductile residue, when the quantity applied was sufficient to produce a dense waterproof surface. It is believed that the ductility of the residue obtained in the manner specified in Table 2 is sufficiently indicative of the adhesive properties of the oil to justify the selection of oils showing a ductile residue for use with noncohesive soils.

A study of Table 5 indicates that with the type and quantity of oil used:

1. Group A–4 subgrades are subject to considerable surface rutting.
2. Groups A–6 and A–7 subgrades furnish good surfaces.
3. Soils which fall on the border line between A–4 and A–6 or A–7 groups give inconsistent results.

The detailed subgrade and condition survey disclosed that, in general, the stability of noncohesive soils (silts) was increased by the addition of oil, and that,

TABLE 5.—*Results of soil tests on the various layers of the different soil types and their reactions with oil*

Soil type	Route on which found	Zone	Mechanical analysis						Physical characteristics of material passing No. 40 sieve							Reaction with oil [1]
			Particles larger than 2 mm.	Particles smaller than 2 mm.							Shrinkage		Moisture equivalent		Group	
				Coarse sand 2.0 to 0.25 mm.	Fine sand 0.25 to 0.05 mm.	Silt 0.05 to 0.005 mm.	Clay smaller than 0.005 mm.	Colloids smaller than 0.001 mm.	Lower liquid limit	Plastic index	Limit	Ratio	Centrifuge	Field		
Putnam silt loam	U. S. Route 63	1	0	3	12	66	19	6	33	9	28	1.6	32	29	A–4	Scaled, checked, pitted.
		3	1	6	12	60	21	11	33	12	29	1.6	34	28	A–4	Do.
		5	0	3	15	42	40	21	64	30	16	1.9	54	52	A–7	Mat intact.
		7	0	3	17	39	41	24	62	37	15	1.9	[2] 60	39	A–6	[3].
		9	0	2	5	60	33	19	52	31	16	1.8	[2] 53	39	A–6	[3].
Uncorrelated type No. 2.	do	1	0	4	15	65	16	5	36	12	25	1.6	29	28	A–4	[3].
		3	0	8	11	57	24	10	41	18	24	1.6	36	33	A–4	[3].
		5	0	2	9	46	43	32	64	32	15	1.8	50	51	A–7	Mat intact.
		7	0	2	8	58	32	16	49	27	16	1.8	[2] 50	34	A–6	Do.
Shelby loam	do	1	0	7	26	54	13	7	34	12	25	1.6	24	31	A–4	[3].
		3	0	5	27	51	17	11	34	14	18	1.8	28	28	A–4	[3].
		5	0	4	15	56	25	10	44	24	17	1.8	31	37	A–7	Mat intact.
		7	2	5	16	53	24	12	41	25	13	1.9	34	28	A–6	Do.
		9	1	3	32	38	26	11	42	27	11	2.0	[2] 44	30	A–6	Do.
Knox silt loam	U. S. Route 65	1	0	1	22	64	13	6	30	8	25	1.6	29	26	A–4	Rutted.
		3	0	1	19	61	19	10	37	17	21	1.7	30	29	A–4	Inconsistent.[4]
		5	0	1	9	77	13	4	39	16	23	1.6	37	30	A–4	Rutted.
		7	0	1	21	68	10	5	39	15	26	1.6	38	32	A–4	[3].
		9	0	0	16	73	11	4	35	12	23	1.6	32	30	A–4	Rutted.
Marshall silt loam	do	1	0	1	15	71	13	8	41	14	27	1.6	36	34	A–4	Inconsistent.
		3	0	0	16	67	17	8	40	17	21	1.7	34	31	[5]	Do.
		5	0	2	14	74	10	8	42	19	23	1.6	36	33	[5]	Do.
		7	0	2	17	70	11	6	42	17	26	1.6	35	33	A–4	Rutted.
		9	0	2	9	80	9	6	35	11	31	1.6	38	30	A–4	Do.
Marshall silt loam deep phase.	do	1	0	2	9	72	17	7	41	17	23	1.7	48	34	A–4	[4] Do.
		3	0	2	16	68	14	7	44	19	22	1.7	39	35	A–7	Inconsistent.
		5	0	2	15	71	12	7	48	23	22	1.7	50	38	A–7	[4] Do.
		7	0	1	18	69	12	6	45	22	23	1.7	39	34	[5]	[3].
		9	0	1	9	75	15	6	43	20	21	1.7	36	32	[5]	[3].
Uncorrelated type No. 3.	do	1	0	2	16	66	16	5	39	15	27	1.6	36	36	A–4	Inconsistent.
		3	0	2	16	61	21	10	54	29	19	1.8	44	40	A–7	Do.
		5	0	4	19	61	16	8	52	31	19	1.7	41	38	A–7	Do.
		7	0	2	16	66	16	5	40	19	20	1.7	45	30	[5]	Rutted.
Marshall silt loam shallow phase.	do	1	0	3	17	69	11	7	36	13	24	1.6	34	28	A–4	[3].
		3	0	3	18	63	16	5	34	15	18	1.8	28	27	[5]	Inconsistent.
		5	0	10	23	57	10	4	33	17	17	1.8	30	23	[5]	Do.
		7	0	10	38	40	12	6	24	11	15	1.9	18	19	A–4	[3].

[1] Reaction with oil is based on behavior of individual zones in cut or fill. The behavior of the various zones when combined with others is not included in this table.
[2] Waterlogged.
[3] No road surface consists of this soil layer. Soil tests results are shown to give complete information for the soil types.
[4] Inconsistent reaction with oil indicates that the condition of the oil-earth mat on this layer varied to such an extent that no condition rating could be applied as generally representative of the performance of this layer.
[5] Blank spaces under group indicate that according to laboratory tests the soils fall between the A–4 and A–6 or A–7 groups.

within the limits of the soil types encountered and with the application of the same type and quantity of oil, the layers of the silty soil types which contained the higher percentages of clay gave the best results. Also, while surface rutting was the most serious failure on the silty soils, it was of minor importance on the clay soils. The failures on the clay subgrades were in the form of surface defects, without any serious loss of stability.

The tendency of the cohesive soils (clays) to give better results than the cohesionless soils (silts) under the conditions and methods of construction may be explained when we realize that we are dealing with a soil made up of an aggregate of certain constituents to

FIGURE 4.—POORLY DRAINED SECTION. THE IRREGULAR SURFACE HAS CAUSED TRAFFIC TO FOLLOW SELECTIVE TRAFFIC LANES. AS A RESULT PORTIONS OF THE SURFACE HAVE BEEN DEPRIVED OF THE IRONING AND KNEEDING EFFECT OF TRAFFIC AND HAVE RAVELED EXTENSIVELY

which we are attempting to impart stability, and that the requisites of stability are a proper combination of internal friction and cohesion. We can do only two things by the addition of oil. One is to stabilize the moisture content by exclusion of as large a percentage as possible, and the other is to impart some cohesion. The cohesive type of soil (clay) already has sufficient cohesion and requires only the exclusion or stabilization of moisture content to furnish stable support; hence it is entirely logical that any waterproofing oil, regardless of its adhesive qualities, will, if and while present in sufficient quantities, serve as an efficient road oil. Such soils as silt or sand lacking cohesion and waterproofing will not supply this lack, although it will eliminate variations in volume due to variations in moisture content. It is therefore logical to expect that the addition of cohesion to this type of soil which can in some degree be added through the use of an oil having adhesive properties, as indicated by the ductility of the residue, will be reflected in more serviceable surfaces.

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Nonuniformity of subgrade soils and degree of consolidation appreciably affected the uniformity of the completed surface. The greater portions of U. S. Highways 63 and 65 on which a detailed soil survey was made consisted of fill over an old road and were composed of mixtures of the various soil layers in different percentages and degree of compaction.

Soil materials which possessed the properties of compaction in a high degree gave the best results and soil material in place reacted better with the oil treatment than the same soil in fill. This was especially true of the soils which were difficult to compact, namely, the silty soils. Of the silty soils, those with some development of field structure were the most compactable. Surface defects (raveling, crumbling, scaling, checking, pitting) were confined almost entirely to fill materials. This fact indicates that more uniform penetration and better final results may be obtained if the subgrade is prepared so that the materials are mixed and consolidated uniformly.

More uniform surfaces and consequently more uniform penetration may be obtained by scarifying the surface to a depth of about 3 inches and thoroughly mixing this loosened material by means of a multiple blade maintainer, or by blading the material back and forth across the roadway. After the soil has been thoroughly mixed it should be distributed uniformly over the surface and carefully maintained during reconsolidation for a period of six weeks prior to the final preparation for treatment.

SURFACE DRAINAGE ESSENTIAL TO GOOD RESULTS

Surface drainage was rated as follows:

0–25____ Side ditches shallow and poorly drained. Road crown insufficient to shed surface water, and water accumulating in slightly depressed areas either penetrated the treated surface or was eliminated by evaporation. Crown less than 0.2 of an inch per foot of width.

25–50____ Side ditches provided satisfactory drainage. Road crown similar to that described above.

50–75____ Side ditches provided satisfactory drainage. Road crown sufficient to shed water readily. Crown between 0.2 and 0.3 of an inch per foot of width.

75–100___ Side ditches provided excellent drainage. Road crown sufficient to shed surface water rapidly and completely, even from slightly depressed areas. Crown in excess of 0.3 of an inch per foot of width.

In observing the ratings as shown on the charted summary under drainage, Table 4, it is noted that with only two exceptions a drainage rating of from 50 to 75 or better was accompanied by a rating, in the fall condition survey, or fair or better. All ratings poorer than 50–75 were accompanied by condition survey ratings of poor or worse. A relationship between efficiency of surface drainage and final result is suggested.

Observations in the field indicate that proper surface drainage is one of the primary requisites of satisfactory results in the oiling of earth surfaces. Figure 4 shows the results of poor drainage. In no case did the department fail to obtain an all-weather roadway during the winter following oiling if ample surface drainage had been provided. In other words, under all soil conditions and within the entire range of oils used, service consistent with the expenditure was obtained if satisfactory drainage was provided.

Many failures were caused by water from rains and melting snow being retained in slight surface depressions for extended periods of time. These failures occurred

most frequently on flat grades which were entirely dependent on direct transverse drainage for the disposal of surface water. It is in this respect only that the character of the profile has any effect on the surface condition. Runoff from the treated portion often caused the untreated sholder to become very unstable and mutilation of the treated surface progressed inwardly from the outer edge.

SUBGRADE MOISTURE AT TIME OF OILING REFLECTED IN SURFACE CONDITION

Reference to Table 4 shows a consistent relationship between the character of surface just prior to treatment, the moisture content and the final results. Figures 5 and 6 illustrate the results obtained under different conditions of moisture. Sections rated as fair or better were free from excess dust and their states of consolidation varied from loose to firm. Surfaces on sections given a final rating of poor or failed, were in general more dusty and in many cases hard and crusted. All sections rated as fair or better in the first condition survey, with the exception of one, show the presence of an appreciable amount of moisture when oiled. The

FIGURE 5.—SECTION IN GOOD CONDITION THREE MONTHS AFTER TREATMENT. THE SURFACE CONTAINED AN APPRECIABLE AMOUNT OF MOISTURE WHEN OIL WAS APPLIED

one section not showing the presence of moisture was rated as poor at the time of the second condition survey. All sections rated as poor or failed, had no appreciable amount of moisture at the time of application of the oil.

If satisfactory surface drainage has been provided so that water will not stand on the surface and be forced into and through the oiled surface by the action of traffic, the next most important requisite to the successful oil treatment of earth is the proper condition of the surface as regards moisture content. For a successful oil treatment it is necessary that the oil penetrate and become intimately mixed with all soil particles by the kneading action of traffic. The mere presence of the proper quantity of oil on or near the surface is not sufficient. All particles must be coated or saturated with oil. The condition of the soil must be such that the oil can enter and mix with all the particles, either by direct penetration or by manipulation and kneading under traffic. The presence of the normal moisture content promotes the conditions just stated in that the moisture keeps open the pore spaces and facilitates the uniform distribution of the oil. By its gradual recession or elimination the moisture promotes uniformity and completeness of the contact of the oil with all particles even though the rate of penetration is retarded to some extent.

FIGURE 6.—SURFACE WAS PARTLY CRUSTED AND HARDENED AT THE TIME OIL WAS APPLIED. SHRINKAGE CRACKS ARE NUMEROUS AND PITTING IS EVIDENT

The character of surface prior to oiling as previously suggested is in a very large measure dependent upon the moisture content. The presence of dust is due to lack of moisture. Most clays exhibit considerable volume change with variations of moisture content. On drying excessive shrinkage takes place resulting in the formation of shrinkage cracks and a hardened crust separated into blocks by the cracks. This is the condition referred to as "hard and crusted" under surface condition rating. The crust has become densified and hardened, and the particles which in a moist condition were separated by films of moisture are now drawn into more intimate contact through the stresses exerted by capillary pressure as the water is eliminated. The major portion of the area of the surface is too dense and hard to receive the oil and as a result it enters the cracks between the surface blocks to remain in concentrated form or to distribute itself in the more moist and consequently more receptive underlayers. The blocks of hardened surface receive some oil by penetration around their surfaces but exhibit a dry and barren interior. If the blocks of crust are not too hard the

kneading of traffic may tend to remedy the condition by helping to redistribute the oil present in the cracks but under extreme conditions the blocks remain hard and unreceptive. The impact of traffic often causes the surface-coated but unpenetrated blocks to be displaced, starting the familiar conditions of raveling and potholing.

Lack of moisture in the case of soils which have lower volume change with loss of moisture, such as the silts or sandy soils, is not so serious, as the pores vacated by moisture are occupied to a considerable extent by air and are open to the entrance of oil. Dust forms rapidly, however, on this type of soil. If this dust is not penetrated so thoroughly as to bind it to the underlying soil, it will cause serious peeling and raveling, and barren spots, insulated by dust layers against penetration, will later pit and rut or ravel.

Owing to weather and traffic conditions the surface to be treated can be maintained in the ideal condition for the reception of oil for only a very short period of time, usually not more than 24 hours.

Of sections included in this study the best results were obtained on those sections which were bladed the same day oil was applied. Blading at this time cut down to the zone of normal moisture content, and the treatment followed before the moisture had received from the exposed surface. Section 2 is an example.

The poorest sections were those which were bladed to finished grade a considerable time before oil was applied. Sections 17 to 21, inclusive, were prepared for oil one month before oil was applied and maintained during this interim by dragging, care being taken not to disturb the compacted portion of the surface. As a result the surface at the time of oiling had practically no crown and portions were dense and crusted.

The outstanding failures of sections studied in this report, excluding failures due to lack of drainage, were due to the conditions suggested above.

WEATHER CONDITIONS EFFECTIVE ONLY IN REGARD TO SUBGRADE CONDITION PRIOR TO OILING

Changes in weather seemed to be effective only to the extent that they affected the moisture content of the surface and consequently its condition prior to oiling.

Within the ranges observed in this study there was no significant effect of air temperature changes except that they might have served to speed or retard changes in the moisture content of the subgrade.

CHARACTER OF PENETRATION MORE SIGNIFICANT THAN DEPTH PENETRATED

Within the range of penetration obtained there did not seem to be any significant relation between depth of penetration and resulting service. It will be noted, however, on reference to Table 4, that the character and rate of penetration seem to be significant. With proper conditions of surface and moisture the penetration was relatively slow and uniform, an exception being a section previously treated. These conditions of penetration coincide with fair or better service ratings. Coinciding with lack of proper amounts of moisture, presence of excess dust, and hardening and cracking of the surface, nonuniform and more rapid penetration was noted. The effect of loose material on the surface prior to oiling is shown in Figure 7. The rapidity of penetration was increased by the presence of cracks. This penetration tended to be selective, however, following the cracks rather than mixing uniformly through the soil. Cores taken so as to include the cracks showed that they had contained considerable quantities of oil, as the soil on each side of a crack was impregnated with oil at depths considerably below the point of maximum penetration entering directly through the surface.

Where the crust was very hard and resistant to penetration, cores taken to the maximum depth of penetration showed a variation in color, grading from a glossy black at the surface to a very light brown at the maximum depth. Nonuniformity and rapidity of penetration, together with selective penetration and variation in color coincide with sections rated as poor or failed.

FIGURE 7.—APPEARANCE OF SURFACE AFTER FIRST APPLICATION OF OIL. NONUNIFORM PENETRATION HAS OCCURRED BECAUSE OF LOOSE MATERIAL ON THE SURFACE. THE PRESENCE OF UNABSORBED OIL ON THE SURFACE IS INDICATED BY THE LIGHT SPOTS

RELATION BETWEEN WIDTH OF UNTREATED ROADWAY ADJACENT TO OILED SECTION AND SURFACE CONDITION

A study of the summary fails to produce any definite information regarding the proper width of untreated surface adjacent to the oiled section. Observations, however, showed that there was a detrimental effect of having untreated sections adjacent to the oiled section, particularly if traffic or the method of maintenance was such as would tend to move untreated earth or dust on to the treated surface.

A development of considerable importance, although it does not have an important effect upon the efficiency of the oiled earth surface as a traffic way, was brought out by observations in connection with these untreated widths in comparison with the treating of full width. This development was the great efficiency of oil treatments as a means of stopping wash or erosion of soils of the noncohesive type such as those derived from loess. In certain areas the Missouri State Highway Department was rapidly approaching the necessity of purchasing additional widths of right of way to obtain earth for

replacement of shoulders removed by erosion. Oil treatment as described in this report immediately and effectively stopped this erosion, saving many dollars in maintenance costs. In extreme cases such simple treatments enabled the holding of fills subject to overflow, which previously had periodically been lost. After oil treatment these fills resisted the water action by virtue of the thin stabilized film or layer which prevented the inception of washing and crumbling. This development leads us to include the factor of conservation of material in our future considerations for rating effectiveness of oil treatment.

EFFECTS OF VARIATION IN TYPE AND QUANTITY OF OIL AND TEMPERATURE OF OIL WHEN APPLIED

No apparent and consistent variation in results due to variation in oil type alone can be noted from the data as herein summarized.

There were, however, some indications that a larger quantity of oil per unit of surface would insure more durable surfaces than were obtained with the use of the original quantity specified, i. e., three-fourths gallon per square yard. Areas of noncohesive soils accidentally subjected at the ends of sections to overlapping treatments from two successive distributor loads invariably showed an improved character of surface over adjoining areas not covered by such overlaps.

On sections treated with No. 4 oil, which was an oil having a ductile residue and a somewhat higher viscosity than oils Nos. 1 and 3, better results were noted than was the case on sections treated with any of the other oils. In connection with the use of this oil it should be noted that very favorable penetration conditions existed.

These indications and the general conviction that somewhat richer surfaces would be beneficial led the highway department to use a larger quantity of a somewhat heavier oil during the following oiling season. In these later applications a total of 1 gallon per square yard instead of three-fourths gallon was used. The principal difference between No. 3 oil (Table 3), and the oil used during 1929 was in their viscosities. That used during 1929 had an average specific viscosity (Engler) at 60° C. of 17, while the No. 3 oil (Table 3) had an average viscosity of 10.82 by the same method of test.

The surfaces treated during 1929 in general were more serviceable and durable than those treated during 1928. This improvement is attributed largely to the increase in quantity and viscosity of the oil, although some improvement was due to better general preparation of surfaces prior to oiling, brought about by increased attention to this item.

No apparent benefit was obtained by raising the temperature of the oil for application above that required to insure against clogging of the distribution nozzles. Increasing the temperature of the oil facilitates and increases run-off from the surface due to decreased viscosity and is detrimental rather than beneficial.

EFFECTS OF VARIOUS TYPES OF TRAFFIC

Mutilative traffic seriously reduced the serviceable life of a considerable mileage of the treated roads, but was localized in effect as a rule and was not considered significant as affecting condition surveys. Figure 8 shows the effect of this type of traffic. The destructive effect was not as noticeable on those surfaces having a live, ductile surface layer. During the winter and early spring, the surface is least resistant to mutilation by traffic and during those seasons much damage is done by the hoofs of cattle and horses. Any reasonable and practical means of mitigating the effects of mutilative traffic will justify some expenditure. As stated previously, in some cases cover material, generally river sand, was applied in a thin layer to the surface. The results seemed to be uniformly beneficial except in the presence of conditions causing the failure of the oil treatment.

FIGURE 8.—MUTILATING EFFECT OF TRAFFIC AFTER RETREATMENT

Sections 16 and 27 indicate clearly that the serviceable life of the treated road is materially shortened if traffic during the first month after application of the oil is not sufficient and of such character as to knead thoroughly and incorporate the oil into the surface. Section 27 was deprived of this traffic, the road having been barricaded during the construction of higher type pavement on an adjoining section. The surface two months after treatment was dusting extensively, the oil being light brown in color and dead, almost entirely devoid of its binding properties. Section 16 is an example of what may be expected when the surface contour is so irregular that traffic uses selective lanes. Several large longitudinal ruts extended over the greater portion of this section when the final preparation was started. These ruts were filled with loose soil and oil applied before this material was compacted. As a result, several major depressions soon appeared where the ruts had been. This condition caused traffic, especially pneumatic tired traffic, to follow selective traffic lanes. The balance of the treated surface soon became dead, dusting and raveling extensively owing to lack of traffic.

METHODS OF MAINTENANCE COMPARED

Sections 10 and 11 are typical examples of the effects of blading a thin layer of earth in from the shoulder, and section 14 is an example of the effects of shaving the treated surface and filling the defects with the material thus obtained.

Both methods, although improving the riding surface temporarily, materially shortened the serviceable life of the treated surface. The dust superimposed on the surface acted somewhat as a blotter, absorbing a portion of the oil, after which it was washed or blown from the surface, carrying this portion with it. The dust also insulated the treated surface against the kneading and solidifying effect of traffic and as a result the oil deadened rapidly. This method of maintenance probably has a more injurious effect when the soil of the oil-earth mat is a silt or silt loam, as this type of soil does not possess appreciable cohesion and is relatively open and porous.

The other method, that of shaving the treated surface and filling the depressions, invariably resulted in considerable scabbing of the surface and the material deposited in the depressions lacked the binding properties necessary to prevent its being whipped out by traffic.

Not only was the life of the roads maintained in this manner shortened, but the surfaces were dusty during dry weather and portions of their length became muddy during wet weather.

THE 1930 OILING PROGRAM

The work and observations outlined in this report gave indications of possible improvements in the treating of earth roads with oil. This resulted in a further revision of practice in oiling earth roads, together with some changes in the road oil specifications during the 1930 season.

The 1930 oiling program consisted of surface treating 287 miles of roads. Of this total mileage 143 miles were re-treated and 144 miles received original treatment. In addition to this mileage, short experimental sections were constructed in accordance with a mixed-in-place method, which involved the mixing of oil with river sand, bank sand, and natural soils.

As regard the specifications, a ductility requirement of over 20 centimeters for the 100 penetration residue was included in the requirements of specification A of Table 2. This change resulted in obtaining an oil with an average ductility of about 30 centimeters. This oil was a topped residual of Arkansas crude with a semi-asphaltic base. The bulk of the oil applied, about 80 per cent, conforms to specification B. This corresponds to oil No. 3 in Table 3, a cracking-process residual of general mid-continent crude, having a ductility of more than 100 centimeters for 100 penetration residue. The revised specifications required oil of both types to have a higher viscosity than formerly, averaging around 16, a greater amount of 100-penetration residue, between 55 and 70 per cent, and allowed for as much as 10 per cent loss after five hours at 163° C.

ORIGINAL TREATMENT AND RE-TREATMENT DESCRIBED

The preparation of the road surface for the reception of the oil continued to be a function of the various local maintenance forces, and the fact that those responsible appreciated the importance of surface preparation and the other conclusions and indications contained in this report was very apparent, and to a considerable extent is reflected in the superior results obtained.

Earth-oiling practice in 1930 as compared with that of previous years included better surface drainage of both roadway and side ditches and more uniform cross-section, free from depressions. In the case of original treatment, road surfaces were prepared so that final preparation was completed immediately before the initial application of oil. The temperature of the oil at the time of application was controlled to prevent run-off on steep grades. The entire width of roadway was treated on those sections composed of loess soil lacking sufficient clay to resist erosion.

The quantity of oil on sections given original treatment was increased from three-fourths gallon per square yard to 1 gallon. This increase was decided upon because previous observations had indicated that the slightly inferior results obtained on soils deficient in cohesion was in a great measure due to their relatively greater porosity, which permitted the penetration of the oil to such a depth that the quantity of oil was insufficient to bind adequately the particles of soil included within the penetration horizon.

The quantity of oil per square yard for re-treatment varied according to the general conditions of the surface soon after the final spring thaw. Some sections were re-treated with one-half gallon per square yard or slightly less, while other sections which were almost completely unserviceable were scarified, bladed, and re-treated, using 1 gallon per square yard.

The number of applications and quantity of each application varied in relation to the total quantity to be applied. Distributions of three-fourths gallon or more per square yard were made in three applications; first application, four to six tenths gallon per square yard; second and third application, two to three tenths gallon per square yard each. Distributions of less than three-fourths and more than four-tenths gallon per square yard were made in two applications, the quantity of each application being designated by the engineer. Distributions of less than four-tenths gallon per square yard were made in one application.

PREPARATION FOR RETREATMENT VARIED ACCORDING TO NEEDS

The preparation of the surface for additional oil varied from a light disking and dragging to complete scarification of the previously treated oil surface. In the northwestern section of the State a considerable mileage of road surfaces was prepared by a light planing of the surface, the loose material thus obtained being utilized in eradicating pits and other surface depressions.

The object of this method was to facilitate the penetration of the new oil by removing the glazed surface and at the same time reclaiming as much as possible the compaction, stability, and waterproofing qualities of the existing mat. This planing was accomplished by means of a special disk planer followed by a multiple-blade maintainer. Each of these pieces of equipment was so constructed that the cutting depth could be accurately controlled. The number of trips necessary to prepare the surface depended upon the type and condition of the existing surface as well as upon the skill of the operator. Care was taken that the bond between the existing oil mat and the underlying soil was not destroyed. The preparation of the old surfaces for retreatment, either by light harrowing or disk planer, both followed by a multiple-blade maintainer to correct irregularities in the surface, almost immediately developed the fact that, if the road thus prepared were subjected to the kneading action of traffic during a series of warm days, the oil would freshen and reassimilate the loosened material, furnishing a smooth surface in excellent condition for retreatment. Figure 9 shows the disk planer and multiple-blade maintainer in operation.

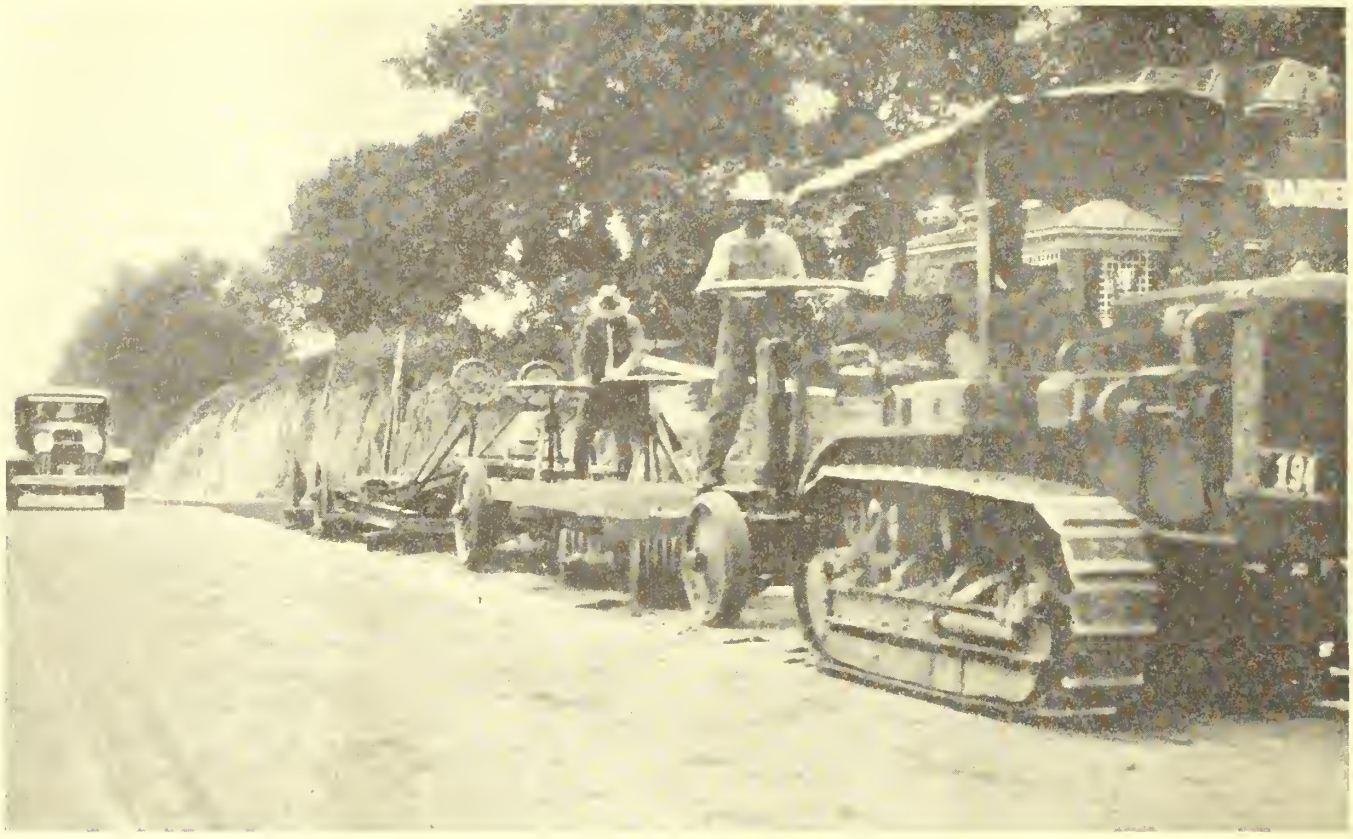


FIGURE 9.—PREPARATION FOR RETREATMENT: TRACTOR HAULING SPECIAL DISK PLANER AND MULTIPLE-BLADE MAINTAINER

Five experimental sections were designed in order to study the feasibility of mixing oil in place with sand or soils. These sections involved three variations in the type of oil used, and three different classes of material were treated. The processing consisted of blading the subgrade to a depth of from 2 to 4 inches, leaving the loosened material in a windrow on the shoulder, applying a primary coat of one-half gallon per square yard to the subgrade, blading the loosened material back and forth, applying oil at the rate of one-half gallon per square yard three or four times, and continuing the blading until the whole mass had a uniform dark brown color. The material was then bladed to a uniform thickness across the roadway and given to traffic for compaction.

The experimental sections are listed as follows:

Section A. Subgrade: A residual soil from weathering of of sandstone; soil group A–3.
Oil: Revised type No. 3 of Table 3.
Section B. Subgrade: A clayey silt; soil group A–5.
Oil: Revised type No. 3 of Table 3.
Section C. Subgrade: A sandy flood-plain soil; soil group A–3.
Oil: Type No. 3 of Table 3 with 70 per cent residue and specific viscosity of 30.
Section D. Subgrade: Fine river sand added to clay subgrade; soil group A–3.
Oil: Topped residual of Arkansas crude.
Section E. Subgrade: A silty clay; soil group A–6.
Oil: Topped residual of Arkansas crude.

CONDITION SURVEY MADE IN SPRING OF 1931

Following the abnormally mild winter of 1930–31 two heavy snowfalls accompanied by freezing weather occurred during the month of March. A condition survey made the first week in April showed all oil-treated earth roads on constructed grades to be in serviceable condition. Re-treated roads on which the multiple-disk planer had been used preliminary to re-treatment were in excellent condition, and it is felt that the type of equipment used for this preparation has demonstrated itself as especially adaptable for the preparation of oiled roads prior to re-treatment. On several of the re-treated jobs a tendency for a mat to form accompanied by corrugation of the mat was noted indicating that one-half gallon per square yard was an excessive application in those cases.

A consideration of weather conditions during the 1930 season develops something more than coincidence. A general drought condition prevailed. It was noted, however, that the portion of the State in which oiling of earth roads was carried on could be divided into three fairly distinct sections according to the amount of rainfall. During five months, May to September, inclusive, the northwestern section had a rainfall 3.95 inches below normal. The rainfall in the north-central section was 6.59 inches below normal and in the west-central section it was 7.92 inches below normal. Without exception the oiled roads in the section having the nearest to normal rainfall were markedly superior to those in the other two sections.

The results obtained on the mixed-in-place sections indicate that a more permanent form of surfacing in proportion to the extra cost involved in additional oil and manipulation can be obtained by developing a mixture as described. Little success was had with oils having a residue of low ductility. No difference was noted in the results obtained from the 55 per cent and the 70 per cent residue. Lack of ductility in the oil residue does not show up as a weakness in the oil when mixed with clay or clayey silts.

Figure 10 shows the condition, in the spring of 1931, of a section consisting of 9-foot concrete on one side and an oiled road on the other.

CONCLUSIONS DRAWN FROM INVESTIGATION

1. Road oil of the types included in this study is an efficient material for use in the treatment of earth surfaces to provide all-weather roads provided applications are made to suitable surfaces, adequately drained, and repeated as needed.

2. The type of oil, as regards basic crude source, is not a major factor, all types yielding fairly satisfactory results with all types of soils, other conditions being favorable.

FIGURE 10.—NINE-FOOT CONCRETE ROAD ON ONE SIDE, OILED EARTH ROAD ON THE OTHER. PHOTOGRAPH TAKEN IN SPRING OF 1931

3. Adequate drainage, both subsurface and surface, is essential to the satisfactory service of oiled earth surfaces. Adequate drainage, on flat topography, requires greater crown than ordinarily constructed on graded earth sections.

4. General profile is not a factor affecting the service of oiled earth surfaces except as it may influence the efficiency of drainage and the character of the subgrade material.

5. To reduce the effects of erosion on noncohesive types of soils, oil should be applied on the full width of roadway.

6. All types of soil included in this study can be efficiently treated with road oil.

7. The soil type is an effective factor because of the physical characteristics of the various soil layers exposed to the application of oil. The physical characteristics of the subgrade soils affect the final results according to whether the soils require only the water-proofing qualities of the oil or in addition require an increase in their cohesive properties. The condition of the surface with respect to dust, hardness of crust, and moisture content at the time of application of the oil is directly reflected in the physical characteristics of the soils. The uniformity of penetration is controlled by the uniformity, texture, and density of the several layers of a soil type.

8. Soils lacking cohesion and inclined to absorb water very readily in quantities sufficient to cause rapid loss of stability (represented in this study by A–4 subgrades), may be more effectively treated with oils having ductile and cohesive bases. Soils which possess cohesion in a high degree and which, when in a stiff or plastic state, do not absorb additional water unless manipulated (represented in this study by A–6 soils), do not require treatment with oils having cohesive bases, as waterproofing without binding will insure fairly satisfactory results. Soils which possess properties from each of the groups mentioned above but can not be placed definitely in either one may give better results if the quantity of oil applied is increased.

9. The character of the surface immediately prior to application of the oil is of major importance. Surfaces to which oil is to be applied should be fairly free from dust and should have pore spaces open to receive the oil. Final preparation of surface for treating should consist of blading to eliminate all dust, crust, and depressions of the road surface, rather than the movement of loosened material to obtain uniform cross-section. A uniformly smooth surface to insure uniform distribution of traffic over the entire roadway is essential for proper development of the surface.

10. The presence of sufficient moisture in the surface is essential in order that the pore spaces be kept open and free to receive the oil. Surfaces free from moisture tend to become dusty and hardened, causing nonuniform and selective penetration.

11. Weather conditions are a factor to the extent that they may influence the moisture content of the surface, the rate of penetration, and the quantity of oil, if loss occurs due to rainfall immediately following application.

12. Within the ranges observed in this study there did not seem to be any significant effect of air temperatures except that they might have served to speed or retard changes in the moisture content of the soil.

13. There was no apparent benefit obtained by raising the temperature of the oil for application above that required for uniform distribution. Increasing the temperature of the oil increased the tendency to flow along or from the surface by decreasing the viscosity of the oil and was detrimental rather than beneficial.

14. A retarded rate of penetration tends to improve the uniformity of distribution of the oil thereby promoting the intimate mixture of oil and soil particles which is desired. Nonuniform penetration, which usually accompanied a rapid rate of penetration, failed to produce the results desired.

15. The presence of untreated earth surfaces adjacent to oiled earth surfaces reduces to a varying extent the effectiveness of oiled surfaces particularly if the untreated surfaces are manipulated, as the untreated earth tends to deaden the treated surfaces.

16. Nonmutilative traffic is highly beneficial to oiled earth surfaces as such traffic tends to knead the oil into intimate contact with the soil particles. This intimate association is a primary requisite for successful treatment of earth with oil.

17. Mutilative traffic seriously impairs the service rendered. Such traffic should be prevented or minimized to the extent possible.

18. Dragging or blading of a good oiled earth surface is harmful. When reshaping is necessary a re-treatment must be given to restore the oiled surface.

THE ACTION OF SULPHATE WATER ON CONCRETE[1]

RECENT TESTS OF SPECIMENS IMMERSED IN MEDICINE LAKE, S. DAK.

Reported by DALTON G. MILLER, Senior Drainage Engineer, United States Bureau of Public Roads, and PHILIP W. MANSON, Division of Agricultura Engineering, University of Minnesota Agricultural Experiment Station, and Minnesota State Department of Drainage and Waters

TESTS of many cylinders stored in Medicine Lake have been made since those reported in Public Roads of October, 1925,[2] and November, 1927.[3] The new tests include some at 3 and 5 years of cylinders from the earlier series and 1 and 3 year tests of cylinders installed in the lake since 1927. The number of cylinders that have been made for the Medicine Lake experiments now totals well in excess of 20,000, of which more than 9,000 have been actually stored in the lake for time periods up to seven years, and nearly 11,000 stored in fresh water in the laboratory for comparison tests up to five years.

These experiments were designed principally to aid in the general improvement of farm drain tile and particularly to develop tile that will endure under the wide range of soil conditions peculiar to Minnesota. The results are applicable, however, to many other sections of the United States and to concrete culvert, water, and sewer pipe exposed to the action of sulphate soils and waters similar to those to which drain tile are subjected.

MEDICINE LAKE

Medicine Lake is a body of clear alkali water located 18 miles northwest of Watertown, S. Dak. Since it has stretches of gravel beach, conditions for installing and examining field specimens are almost ideal. Analyses of water samples collected at different seasons of the year have shown a total salt content that ranges between 2.34 and 4.72 per cent, consisting almost entirely of the sulphates of magnesium and sodium. An average of four analyses taken December 10, 1923, February 14, 1924, April 29, 1924, and July 1, 1925, is given in Table 1. Medicine Lake freezes over during winter months, but all cylinders of these experiments have been installed at depths well below any frost action, as it was not desired to introduce this variable.

TABLE 1.—*Average of four analyses of water from Medicine Lake, S. Dak.*[1]

RADICALS								
Na (Calc)	Ca	Mg	NO_3	Cl	SO_4	CO_3	HCO_3	Total
MILLIGRAMS PER LITER (PARTS PER 1,000,000)								
3,036	717	5,079	1	509	27,021	88	313	36,764
PERCENTAGE REACTING VALUES								
10.38	2.93	36.69	0.01	1.01	48.22	0.30	0.46	100.00

[1] Analyses by the water and beverage laboratory, Bureau of Chemistry and Soils, U. S. Department of Agriculture.

CONCRETE TEST CYLINDERS USED

Two by four inch cylinders have mostly been used for a number of reasons, among which is the fact that the 2-inch diameter roughly approximated the thickness of the walls of many of the tile used in public ditches of Minnesota and other States of the Middle West. Special attention is directed to the fact that these cylinders, while small, have been for the most part made of concrete, not mortar, although in none have there been used pebbles coarser than three-eighths inch. The aggregate passed all standard physical tests and was separated into screen sizes and recombined to produce a fineness modulus of 4.67.

The cylinders, with some exceptions, were made in batches of nine, of a 1:3 mix, with a relative consistency of 1.00, and a water-cement ratio averaging about 0.62. They were cured the first 24 hours in a moist closet at room temperatures. Such mortar cylinders as were made of standard Ottawa sand fairly well represent very poorly graded aggregate such as is too frequently used in smaller size tile.

No attempt has been made in this work to show the direct influence of aggregate grading, water-cement ratio, quantity of cement in the mix, and those other factors well recognized as greatly affecting the 28-day strength. Instead a 1:3 concrete with the highest unit strength and lowest absorption obtainable for the particular aggregate, within the limits of grading permissible for 2 by 4 inch cylinders, was adopted as the laboratory standard. It was assumed that, for any given set of conditions, concrete with the highest 28-day strength is most resistant to disintegration and, therefore, that comparative tests to show the influence of a single variable, using this type of concrete, would give the most consistent results possible.

Results of all tests are recorded in Table 2; and in order to group properly the essential data for convenient study, they have been divided into six parts, as follows:

Part 1.—Curing in water vapor or steam between temperatures of 100° and 350° F.
Part 2.—Portland cements from different mills
Part 3.—Special cements other than high alumina
Part 4.—High alumina cements
Part 5.—Surface treatment or impregnation
Part 6.—Admixtures.

A few series naturally fell under more than one heading and have been so listed. In the following pages the results of these six studies are discussed.

CURING IN WATER VAPOR EFFECTIVE AT HIGH TEMPERATURES

A total of 5,850 cylinders in 130 series were cured in water vapor or steam at temperatures of 100°, 155°, 190°, 212°, 230°, 260°, 285°, 315°, and 350° F. for time periods, at most temperatures, ranging between 45 minutes and eight days. The data obtained from 15 series cured in air and water only are also given in Table 2, Part 1, for purposes of comparison. One-year tests have now been completed for all 145 series and 5-year tests have been completed for 83 of them. The only series in which 5-year tests have not been made are those cured at temperatures of 315° and 350° F.

A number of points of interest have developed as a result of this phase of the work, many of which have been previously discussed [4] in detail and will not be

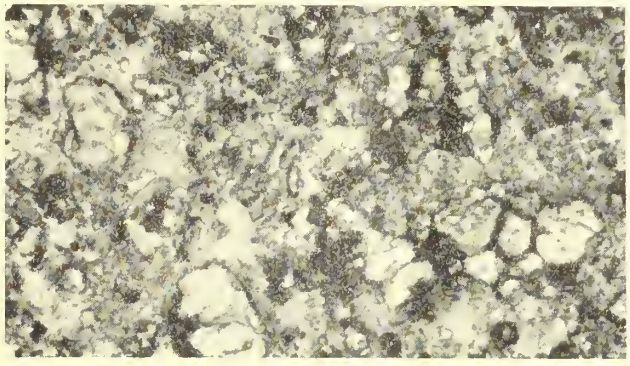
[1] University of Minnesota Paper No. 1012, Journal Series. This paper is a report of progress of experiments at University Farm, St. Paul, Minn., in the drain tile laboratory conducted by the department of agriculture of the University of Minnesota, the department of drainage and waters of the State of Minnesota, and the U. S. Department of Agriculture.

[2] The Action of Sulphate Warer on Concrete, Public Roads, vol. 6, No. 8, October 1925, p. 174.

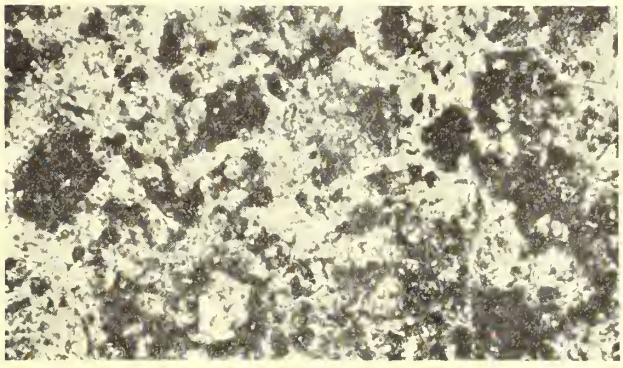
[3] The action of Sulphate Water on Concrete, Public Roads, vol. 8, No. 9, November, 1927, p. 203.

[4] Strength and Resistance to Sulphate Waters of Concrete Cured in Water Vapor at Temperatures Between 100 and 350° F., by Dalton G. Miller, Proceedings, Am. Soc. Testing Mats. vol. 30, Part II, 1930, p. 636.

repeated here. It was noted, among other things, that 7-day compression test results of cylinders cured at a temperature of 315° F. were somewhat more erratic than were those for cylinders cured at other temperatures between 100° and 350° F. It is not yet possible to formulate an opinion as to the significance of this phenomenon as related to the resistance to disintegration of concrete in the presence of sulphate waters, for the reason that tests of cylinders at these higher temperatures have been made only after one year's exposure. Study of the data of Part 1, Table 2, shows that the greatest range in strength ratio values at one year was given by the cylinders cured at 315° F. in series 841–849



Neat cement briquet cured 12 days in water at room temperature



Neat cement briquet cured 24 hours in steam at 212° F.

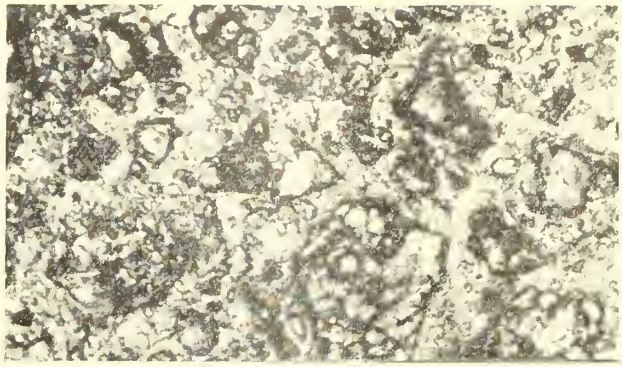
FIGURE 1.—PHOTOMICROGRAPHS OF NEAT CEMENT BRIQUETS SHOWING PHYSICAL CHANGES RESULTING FROM DIFFERENT CONDITIONS OF CURING. NOTE THAT HYDRATION HAS BEEN MUCH MORE NEARLY COMPLETE IN THE STEAM-CURED BRIQUET

but, regardless of this, the following general conclusions have been drawn, based on the tests to date.

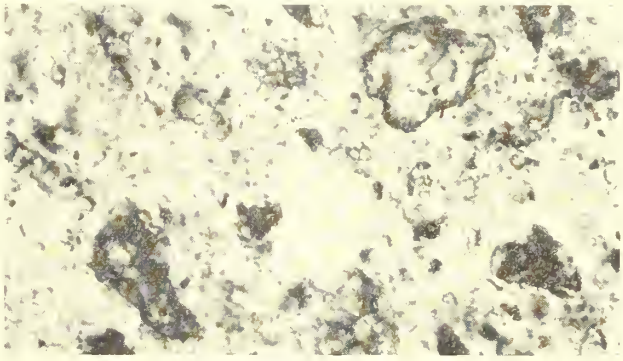
1. Curing in water vapor at temperatures between 100° and 190° F. did not generally increase resistance of concrete to the action of sulphate waters; on the contrary, in some cases a decrease was indicated. An exception to this occurred where certain admixtures were used, as later discussed.

2. Curing concrete in water vapor at temperatures of 212° F. and upward markedly increased resistance to the action of sulphate waters with the data definitely indicating increase of resistance with increase of curing temperatures between 212° and 285° F. for a 12-hour curing period.

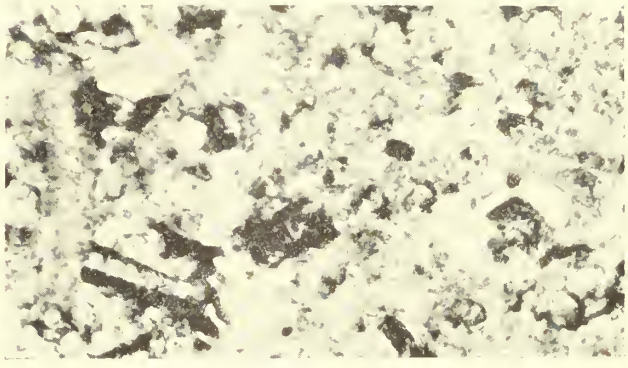
3. Curing concrete in water vapor at 212° F. has been more effective in developing resistance to the action of sulphate waters when continued for six days than when continued but two days.



Neat cement briquet cured 12 days in water at room temperatures



Neat cement briquet cured 24 hours in steam at 212° F.



Neat cement cylinder stored 8 years in water

FIGURE 2.—PHOTOMICROGRAPHS OF NEAT CEMENT SPECIMENS SHOWING PHYSICAL CHANGES RESULTING FROM DIFFERENT CONDITIONS OF CURING. NOTE THAT HYDRATION HAS BEEN LEAST COMPLETE IN THE 12-DAY WATER CURED BRIQUET AND ABOUT EQUALLY COMPLETE IN THE 24-HOUR STEAM-CURED BRIQUET AND THE 8-YEAR WATER-CURED CYLINDER

4. Until more 5-year tests are completed for specimens cured at the higher temperatures it is not possible at this time closely to correlate curing temperatures and lengths of curing period with resistance to sulphate waters of concrete cured in water vapor at all temperatures between 100° and 350° F. It is significant, though, to note that for temperatures between 100° and 285° F. those specimens cured at the highest temperatures and for the longest periods have made the most favorable showings.

5. Regardless of whatever chemical or physical changes in Portland cement increase resistance, following curing in water vapor at temperatures of 212° F. and upward, hydration of the cement grains is very greatly accelerated. This fact is well illustrated in Figures 1 and 2 by the photomicrographs of thin sec-

tions of specimens in which it may be seen that the steam-cured cement grains are greatly altered and reduced in size, within a few hours, to a degree only approached in the water-cured specimens after seven years, as shown in Figure 2. Portland cement I of Part 2, Table 2, was used in the briquets of Figure 1 while equal parts of cements A and B were used in the specimens of Figure 2. Figures 3 and 4 indicate the trend of results from this part of the investigation.

SUPERIORITY OF CERTAIN CEMENTS INDICATED BY RESULTS FROM DIFFERENT MILLS

There were 5,400 cylinders in 120 series in which were used standard Portland cements from 34 different mills. One-year tests have been completed for all 120 series and 5-year tests have been made for 76 series, including 73 series that have completely failed at five years. Giving due weight to those cylinders that have disintegrated, and considering such compression tests as have been made, conclusions relative to this group may be stated as follows:

1. Enough difference exists in the resistance of cylinders of standard Portland cements from different mills to justify specifying particular cements for concrete that must withstand such conditions and also to warrant further investigation regarding the constitution and manufacture of the cements found best qualified to withstand alkali action. The marked differences in resistance of these cements are evidenced by appearances of the cylinders at all ages, by such 3 and 5 year compression tests as have been made, and by actual comparative strength ratios at one year.

2. The resistance of different lots of cement from any plant is fairly constant.

Figure 5 illustrates the variations in resistant properties of cements from different mills.

SPECIAL CEMENTS SHOW LITTLE SUPERIORITY OVER STANDARD BRANDS

Eighteen hundred cylinders fall into this group of 40 series in which were used 12 special cements, and, for comparison tests, 6 standard Portland cements. These cements include the five high early strength cements listed as D, E, F, H, and I, each of the last four of which was tested parallel with its companion Portland cement AA, BB, CC, and I, in the order named; the special cement G, which is Portland cement X specially factory treated for waterproofing qualities; cement B, a reground Portland cement; cement C, which is the same as B with an addition of a carborundrum preparation added during regrinding; cements A1, A2, and A3, which are standard Portland cements to which has been added during the grinding process gypsum treated with tannic acid, of which A2 is standard Portland cement Y, treated; and special cement X, which is an imported mason's cement containing about 30 per cent of diatomaceous silica mixed with the cement clinker before grinding.

None of the 12 special cements forming the basis of Part 3, Table 2 have shown sufficient increased resistance to warrant specifying them for concrete to be exposed to the action of sulphate water. Although it is doubtful, completion of the 5-year tests may possibly make necessary a revision of this statement. The results of tests with special cements are shown in Figure 6.

HIGH ALUMINA CEMENTS SHOW REMARKABLY HIGH RESISTANCE

One American and two French manufactured high alumina cements were used in the experiments for which were made 2,520 concrete and Ottawa sand cylinders in 56 series, covering a rather wide range of curing conditions, a number of mixes, and combinations in different proportions with standard Portland cement. One-year, 3-year, and 5-year compression tests have been completed for cylinders from all 56 series.

These tests have developed the surprising fact that, excepting for the 1:5 mix standard Ottawa sand cylinders of series 280 and 292, the very wet mixed cylinders of series 400, and the cylinders of series 435, 436, and 437, in which were used combinations of high alumina and standard Portland cement, the high alumina cement cylinders in Medicine Lake generally had higher compressive strength at one year than did the check cylinders stored in tap water in the laboratory. At five years the difference in favor of the Medicine Lake cylinders was even more pronounced than at one year as shown by strength ratios of 151 per cent, or greater, for the cylinders of 24 series and of 191 per cent or greater for the cylinders of six series, with a high value of 272 per cent for the cylinders of one series. These high strength ratios are not the result of abnormally high compression tests of the Medicine Lake cylinders at one and five years, but are due to the fact that while the Medicine Lake cylinders increased in strength following the 28-day tests, the check cylinders stored in the tap water in the laboratory decreased in strength. No completely satisfactory explanation can be offered for this. It is known, however, that the reaction to temperature of high alumina cement is considerably different from that of standard Portland cement; and it is entirely probable that the comparatively low temperature of the water of Medicine Lake has afforded conditions more favorable for high alumina cement concrete than has the warmer tap water in the laboratory tank. The following facts, brought out by collateral tests conducted by the laboratory tend to support this view.

(*a*) High alumina cement cylinders buried 4 to 5 feet deep in neutral soils have consistently increased in strength up to five years, with no tests yet available for longer periods.

(*b*) High alumina cement cylinders stored in the laboratory in 1 per cent solutions of sodium sulphate have shown no tendency to test stronger at any age than check cylinders stored in tap water in the same room.

(*c*) High alumina cement cylinders stored in tap water in the laboratory have, in several instances, increased in volume, as measured, as much as 0.6 of 1 per cent. Portland cement cylinders under identical storage conditions have rarely increased in volume as much as 0.1 of 1 per cent, and where, following storage in sulphate solutions, they have increased in volume as much as 0.6 of 1 per cent the compressive strength has been only about 50 per cent of normal.

The general behavior of the high alumina cement cylinders stored in Medicine Lake after periods of exposure up to 6½ years, are briefly summarized in the following paragraph. It should be noted that none of these cylinders have been exposed to frost action, to alternate wetting and drying, or to other than very mild temperatures during the most of the year. Experience has shown that the performance of concrete made with high alumina cement depends to a marked degree on the curing of the concrete, and it should be noted that these specimens were carefully cured.

High alumina cement cylinders stored in the sulphate water of Medicine Lake have displayed remarkable resistance to disintegration as indicated by compression tests up to five years and by visual appearance up to six and one-half years. So pronounced has been this resistance that cylinders from 34 series of a total of 53 series made with only high alumina cement showed higher crushing strength after five years in the lake than they did at one year, while the loss of strength was less than 9 per cent for 10 of the remaining 19 series. Of the other 9, the cylinders of series 142, 147, and 156, which had no air hardening previous to exposure, showed respective strength losses of 10, 15, and 12 per cent; those of series 291 of 1:5 concrete showed a loss of strength of 22 per cent; those of series 254–255 of 1:3 standard Ottawa sand mortar showed a strength loss of 27 per cent. Cylinders of series 280, 292, and 400 were the only ones that have shown any appreciable visual evidence of deleterious action. Cylinders of series 280 and 292 were lean-mixed, 1:5 mortar of standard Ottawa sand with high water cement ratios, and failed the first year. The very wet mixed 1:3 concrete cylinders of series 400 gave a fairly satisfactory 3-year test, although some of the cylinders were then showing considerable action. It is evident from these results that, within the limits of exposure represented by Medicine Lake, the behavior of high alumina cement has, to date, very nearly approached the ideal. Figures 7 and 8 show the excellent results obtained with this material.

SURFACE TREATMENT OR IMPREGNATION GIVES FAIR RESULTS

Comparatively few tests are listed under this heading as this group consists of but 810 cylinders in 18 series, of which 5 are untreated series included for purposes of comparison. Experiments were conducted with but the four products, Inertol, linseed oil, McEverlast, and sulphur. One-year tests have been completed for all 18 series, 3-year tests for 11 series, and 5-year tests for 6 series. On the basis of these limited data the results are very generally summarized as follows:

1. Inertol-tested Medicine Lake cylinders gave 86 per cent of normal strength at 5 years. It is evident that Inertol has appreciably retarded deterioration of the concrete.

2. Linseed oil treated Medicine Lake cylinders of 4 series averaged 97 per cent normal strength at one year and were 95 per cent as strong at three years as at one year.

3. McEverlast-coated Medicine Lake cylinders of 6 series averaged 93 per cent of normal strength at one year.

4. The sulphur treatment accorded the cylinders of series 294 gave negative results as the treated cylinders failed more quickly than did the untreated ones of series 293.

The results obtained with surface-treated or impregnated cylinders are shown in Figure 9.

SOME ADMIXTURES SHOW SLIGHT POSITIVE EFFECT

This is the third largest of the six groups of cylinders discussed in this report and under this heading are the records of 5,031 cylinders in 120 series, of which 26 are series in which no admixture was used, included for purposes of comparison. In the remaining 94 series the following 19 admixtures were used: Alkagel A, barium chloride, Barnsdall Admix. blast furnace slag, Cal, calcium chloride, Celite, Colloy, Earthcrete, powdered fuel ash, Ironite, kerosene, Medusa Waterproofing, oil, Omicron, sulphur, Trass, Truscon, and volcanic ash. Each admixture was added to the batch as an extra ingredient, and, when necessary, the quantity of mixing water was increased to produce the desired workability. The amount of admixture used is expressed, in all cases, as a percentage of the dry weight of the cement. The relatively high temperatures of 100°, 155°, and 212° F. were introduced in the curing of cylinders of certain admixture groups, and, in view of some of the interesting developments, it is regretted that all these temperatures were not used with each of the admixtures as, generally speaking, admixtures in normally cured concrete have not been particularly encouraging.

Most of the admixtures, except as the quantities became excessive, did not greatly influence the compressive strength and absorption of the concrete, although the four admixtures, kerosene, Medusa Waterproofing, oil, and Truscon Waterproofing Paste very definitely reduced absorption. It is not yet known as to what effect on resistance to disintegration this reduction in absorption has had in the case of the kerosene and oil-mixed concretes; but the data of Table 2, Part 6 relating to the other two products clearly show that lowering the absorption of concrete does not, in itself, necessarily increase resistance to the sulphates of sodium and magnesium.

In considering the results obtained with admixtures it is well to keep in mind that in all cases the Portland cement used was a combination of two brands, neither of which had displayed a high degree of resistance to the sulphates of magnesium and sodium. It is believed the test results are truly comparable for the different admixtures but it is admittedly possible that using a more resistant cement might have better emphasized any merit, if slight, actually possessed by a particular product. In other words, starting with a resistant cement, anything that increased resistance of the concrete might have displayed considerable better net value than the same treatment accorded concrete in which was used a cement of low resistance. There follows a brief summary of results obtained with each admixture.

1. Alkagel A is the trade name of a colloidal paste of copper and iron soaps, together with paraffin, which smells strongly of ammonia and loses 81 per cent of its weight on drying. Under the condition of these tests this product proved valueless.

2. Barium chloride was without value when 6 per cent was used; and 12 per cent was only of slight value.

3. Barnsdall Admix is described as a "pure, finely ground, meta-colloidal Tripoli silica." According to published analyses it is about 97 per cent silica. It was used in quantities varying from 3.75 to 30 per cent and in 1:3, 1:2:3, and 1:2:4 concretes. Two aggregates and three brands of Portland cement were used in these experiments. This material does not appear to have appreciably influenced the resistance of any of the specimens in which it was used, as indicated by tests at one year.

4. Blast furnace slag ground, screened and recombined in proportions of one part passing the No. 100, and retained on the No. 200 sieve, and five parts passing the No. 200 sieve, slightly increased resistance when 10 per cent was used, and gave nominally better results when 40 per cent was used, in normally cured cyclinders. The cylinders of series 161 cured 48 hours at 155° F. in which was used 40 per cent of blast furnace slag, tested 48 per cent of normal strength at five years. These were the only cylinders of this group that showed real improvement.

5. Cal is a material obtained by pulverizing the dried or undried product resulting from a mixture of either quicklime or hydrated lime, calcium chloride, and water.[5] Additions of 4 or 8 per cent to cylinders normally cured slightly increased resistance with little in favor of 8 per cent. Additions of 4 and 8 per cent to cylinders cured at 155° F. markedly increased resistance, giving 5-year strength ratios of 82 and 93 per cent, respectively.

6. Calcium chloride gave substantially the same results as did Cal, except that 4 per cent made slightly better showings than did 8 per cent. Additions of 4 and 8 per cent to cylinders cured at 155° F. gave 5-year strength ratios of 84 and 82 per cent, respectively.

7. Celite is a diatomaceous silica that did not appreciably increase the resistance of any of the cylinders in which it was used.

8. Colloy is a fine grained siliceous material containing about 30 per cent alumina. It did not appreciably increase the resistance of any of the cylinders in which it was used.

9. Earthcrete is a powder that, used in proportions of 0.27 and 1.06 per cent, did not appreciably increase the resistance of any of the cylinders in which it was used.

10. Powdered fuel ash used in the proportion of 2½, 5, and 10 per cent did not increase the resistance of any of the cylinders in which it was used.

11. Ironite is the trade name for a finely ground product of heavy unit weight, consisting largely of iron and iron oxide with some ammonium chloride. Twenty per cent of it appreciably increased the resistance of three series of cylinders in which it was used. A strength ratio of 27 per cent at five years was obtained for normal curing conditions, 87 per cent when cured at 100° F., and 94 per cent when cured at 155° F.

12. Kerosene used in quantities of 1, 2, 4, and 8 per cent appreciably reduced absorption of the cylinders; but, in proportions under 8 per cent, it had slight effect on the 28-day strength. Use of 8 per cent decreased the strength. Its effect on resistance to disintegration has not yet been determined.

13. Medusa Waterproofing, a powdered product used in proportions of 1, 2, and 4 per cent, has not increased resistance, if the tests at one year may be taken as conclusive. This is true in spite of the fact that absorption of the cylinders decreased as the quantity of the admixture was increased.

14. Medium grade automobile oil, used in quantities of 1, 2, 4, and 8 per cent, greatly reduced absorption of the cylinders, but, in proportions under 8 per cent, it had slight effect on the 28-day strength. Use of 8 per cent decreased the strength. Its effect on resistance to distintegration has not yet been determined.

15. Omicron, a powdered product used in proportions of 3.75 and 7.5 per cent, did not appreciably increase resistance. When used in proportions of 15 and 30 per cent it made fairly favorable showings at one year with strength ratios of 62 and 89 per cent, respectively, although these cylinders show considerable action. (See fig. 12, left.)

16. Sulphur used in proportion of 10 per cent gave negative results.

17. Trass is a fine silicous material of volcanic origin, containing some 20 per cent alumina which, added in proportions of 33 and 66 per cent to cylinders mixed 1:3, 1:4, and 1:5, somewhat retarded disintegration as indicated by tests at one, three, and five years.

18. Truscon Waterproofing Paste, concentrated, added in proportions of 1, 2, and 4 per cent, did not appreciably increase the resistance of any of the cylinders in which it was used, although absorption of the cylinders decreased as the quantity of the admixture was increased. Those cylinders with 4 per cent of Truscon had an absorption only 59 per cent that of cylinders without any admixture.

19. Volcanic ash from west-central Nebraska added in proportion of 20 per cent did not appreciably increase the resistance of those cylinders normally cured. The results were only slightly more favorable for the cylinders cured at temperatures of 100° and 155° F.

Figures 10, 11, and 12 illustrate the results obtained from the use of the various admixtures.

[5] Technologic Paper No. 174, U. S. Bureau of Standards.

RESULTS OF INVESTIGATION SUMMARIZED

The essentials of the foregoing conclusions as to the influence of the various factors considered, on the resistance of concrete to the sulphate water of Medicine Lake, are condensed into the following statements:

1. Resistance has been markedly increased by curing in water vapor at temperatures of 212° F. and upward, almost to the point of immunity to action for the most favorable temperatures and curing periods.

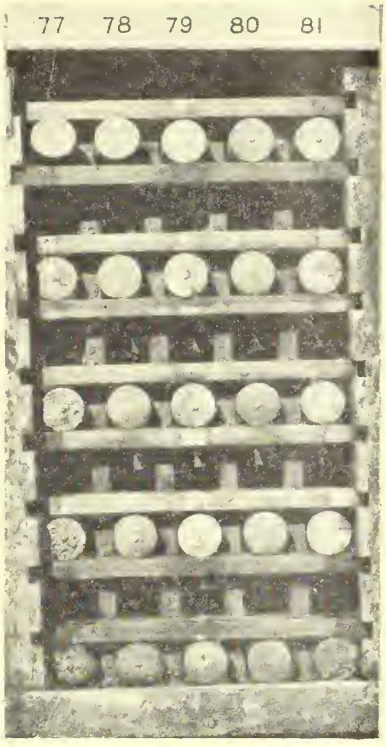
2. Enough difference exists in the resistance of standard Portland cements from different mills to justify specifying particular cements for concrete to be subjected to these special conditions of exposure.

3. Special cements, other than high alumina, have been of little or no value.

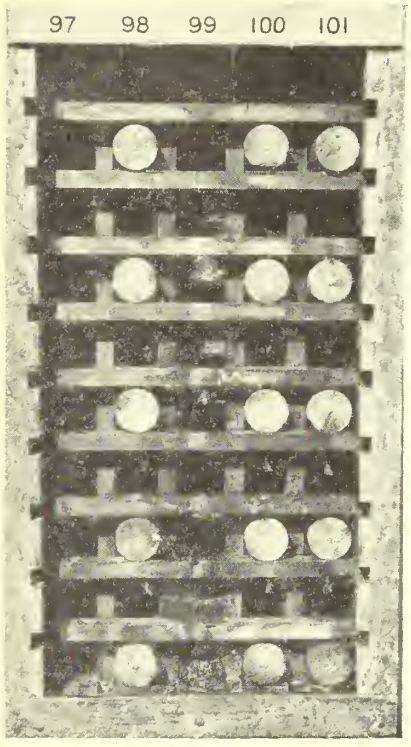
4. Within the limits of exposure conditions represented by these tests, the behavior of high alumina cement has to date very nearly approached the ideal.

5. From the results of tests up to three years it appears that surface treatments of linseed oil have been of considerable value, with indications that later tests may be less favorable. The Interol treatment was of appreciable value up to five years. The McEverlast treatment showed up favorably at one year, but displayed surface indications of less favorable results at three years and later.

6. Only the four admixtures, Cal, calcium chloride, Ironite, and Trass, appreciably retarded action on normally cured concrete, and these to very limited degrees. The use, however, of Cal, calcium chloride, and Ironite in cylinders cured 48 hours in water vapor at temperatures of 155° F. increased resistance to such a degree that at five years cylinders containing 8 per cent of Cal had 93 per cent normal strength, those containing 4 per cent calcium chloride had 84 per cent, and those containing 20 per cent Ironite had 94 per cent. Cylinders of the latter type cured at 100° F. had 87 per cent of normal strength at five years. Some benefit was also noted by curing at 100° and 155° F. cylinders containing 20 per cent of volcanic ash. Appreciably increased resistance following curing at temperatures of 100° and 155° F. has only been noted in the cylinders containing these admixtures. This suggests the practical possibility of a combination of certain admixtures and moderately high curing temperatures for concrete products to be subjected to the action of sulphate waters.



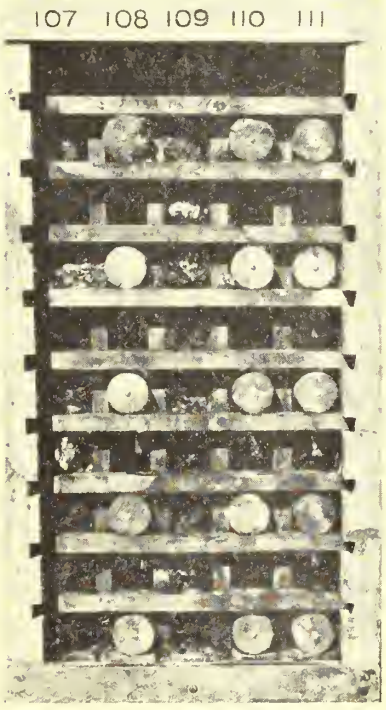
Cylinders cured in steam at 212° F., after six and one-half years in lake



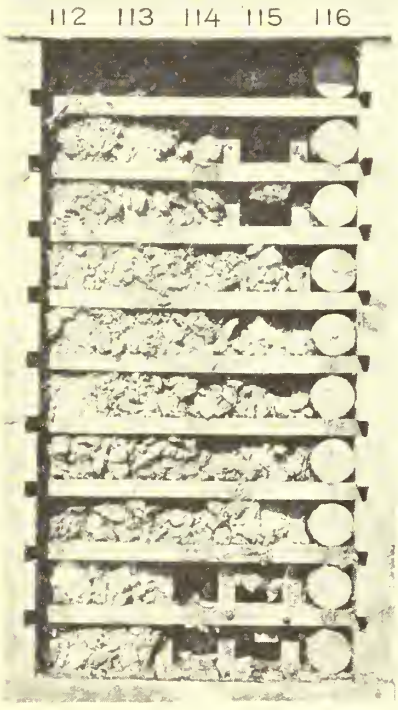
Cylinders cured in water vapor at 155° F., after six and one-half years in lake. Series 98, 100 and 101 received additional curing in steam at 212° F.

Cylinders cured in water vapor at 155° F., after six and one-half years in lake. Series 103, 105 and 106 received additional curing in steam at 212° F.

FIGURE 3.—CYLINDERS OF VARIOUS SERIES CURED IN WATER VAPOR OR STEAM, AFTER SIX AND ONE-HALF YEARS IN LAKE. ALL THE CYLINDERS IN ANY VERTICAL ROW ARE OF THE SAME SERIES AND THE SERIES NUMBERS IS SHOWN AT THE TOP

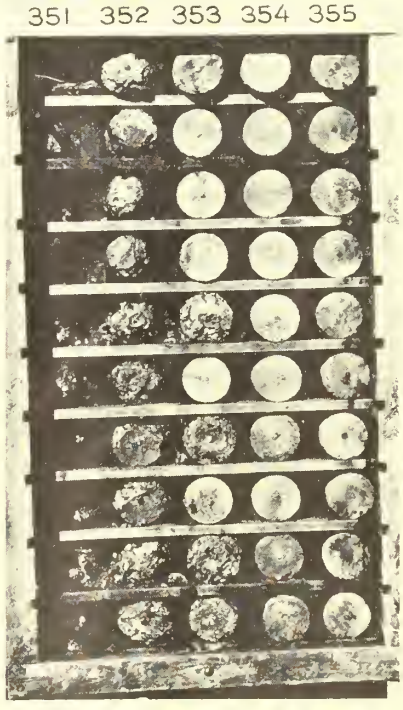


Ottawa sand mortar cylinders cured in water vapor at 155° F., after six and one-half years in lake. Series 108, 110, and 111 received additional curing in steam at 212° F.



Ottawa sand mortar cylinders cured as follows, after four years in lake:

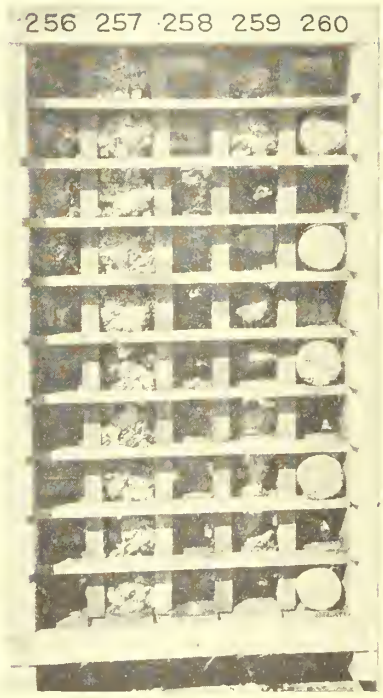
Series 112.—Moist closet, 24 hours, tap water 27 days
Series 113.—Moist closet, 72 hours, air 25 days
Series 114.—Moist closet, 24 hours, water vapor at 100° F., 48 hours, air 25 days
Series 115.—Moist closet, 24 hours, water vapor at 155° F., 48 hours, air 25 days
Series 116.—Moist closet, 24 hours, steam at 212° F., 48 hours, air 25 days



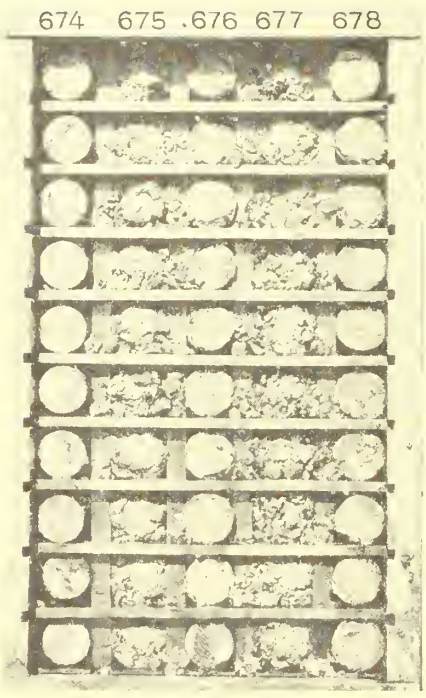
Cylinders cured in water vapor or steam for 12 hours at the following temperatures, after five years in lake:

Series 351.—190° F.
Series 352.—212° F.
Series 353.—235° F.
Series 354.—260° F.
Series 355.—285° F.

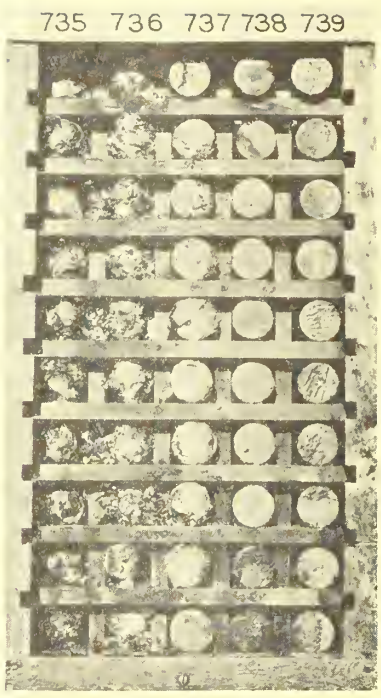
FIGURE 4.—CYLINDERS ILLUSTRATING THE EFFECTS OF CURING IN WATER VAPOR AND STEAM



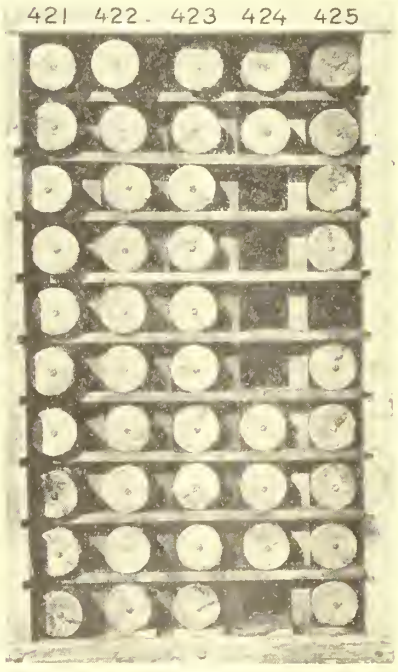
Cylinders made from different brands of standard Portland cement, after five and one-half years in lake



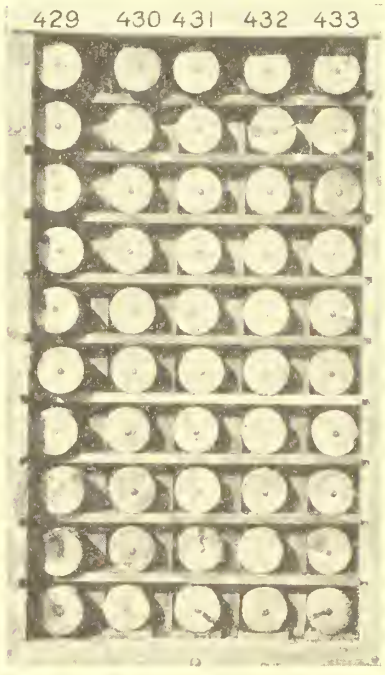
Cylinders made from different brands of standard Portland cement and cured in distilled water 20 days, air 35 days, after three years in lake



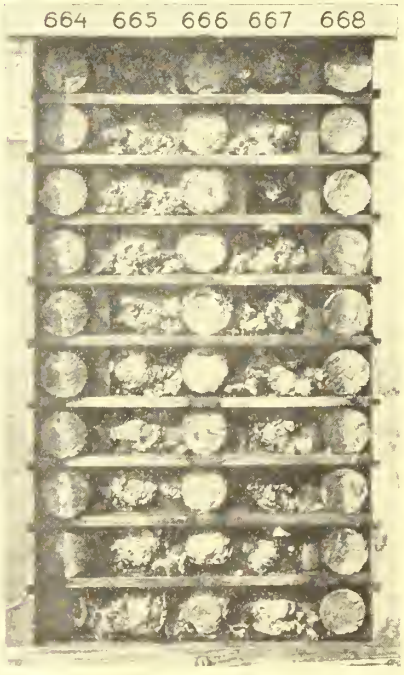
Cylinders made from different brands of standard Portland cement and cured in distilled water 20 days, air 35 days, after two years in lake



Neat cement cylinders made from different brands of standard Portland cement, after four years in lake

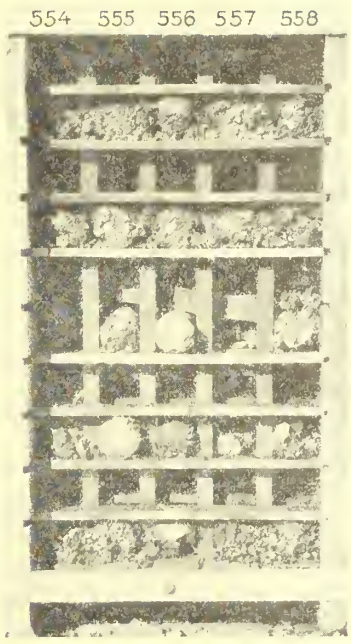


Series 429, neat cement cylinders made from high alumina cement B; series 430–433, concrete cylinders made from different brands of Portland cement, after four years in lake

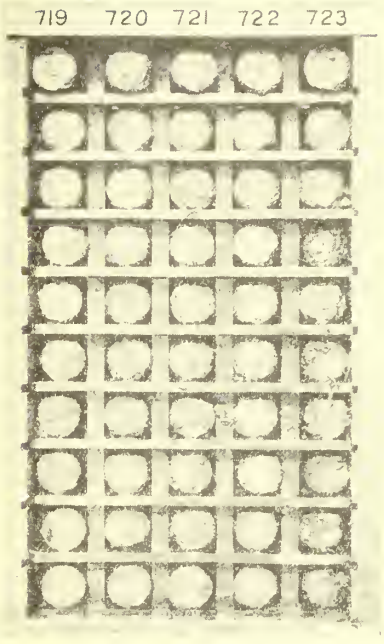


Cylinders of different brands of standard Portland cement cured in distilled water 20 days, after three years in lake

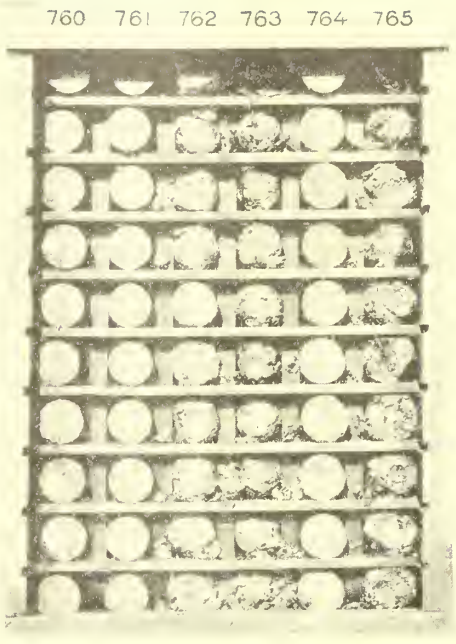
FIGURE 5. COMPARISON OF CYLINDERS MADE FROM DIFFERENT BRANDS OF PORTLAND CEMENT



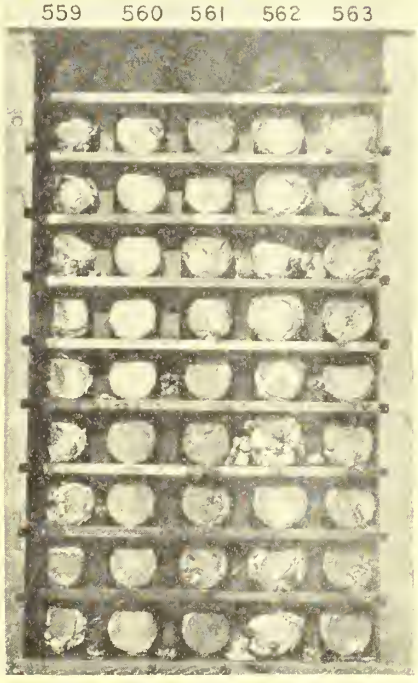
Cylinders made up as follows, after three years in lake:
Series 554—One-half Portland A, one-half Portland B
Series 555 and 557.—Portland Y
Series 556 and 558.—Special A2



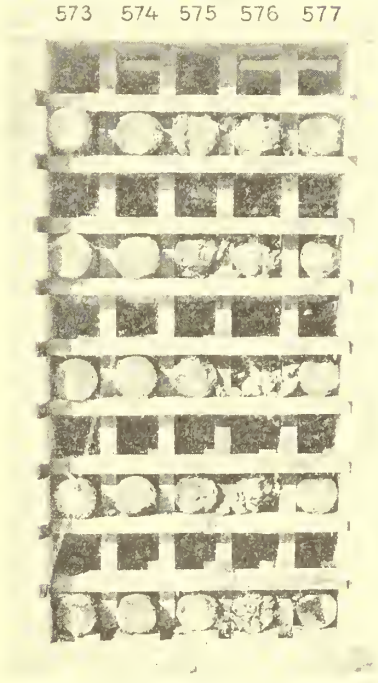
Cylinders made up in following proportions by volume, after three years in lake: Series 719—1:0.94; series 720—1:1.88; series 721—1:1.88; series 722—1:2.82; series 723—1:4.70



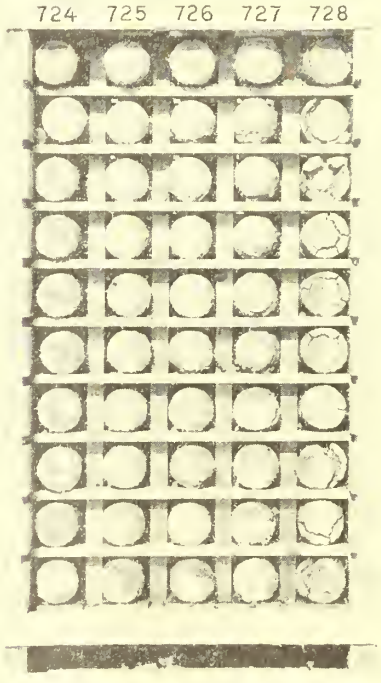
Cylinders of series 761, 763, and 765 made from standard Portland cement; series 760, 762, and 764 made from special cements from the same mills, respectively. After one and one-half years in lake



Series 559, 560, and 562, sand cylinders made from standard Portland cement; series 561 and 563, sand cylinders made from special cement A2. After two years in lake

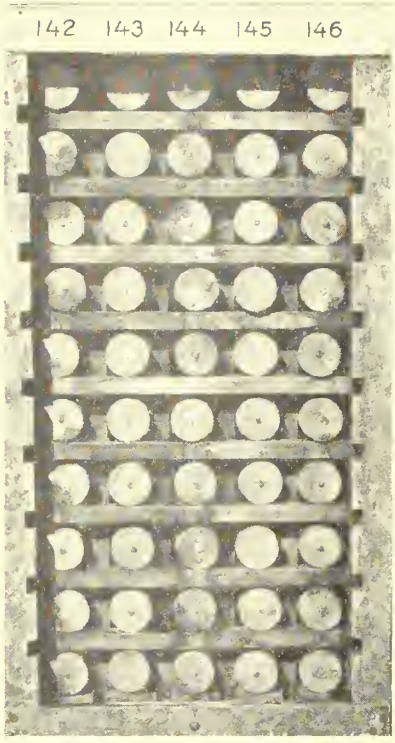


Series 573 and 574 made from special cements; series 575 to 577 made from standard Portland cements. After three years in lake

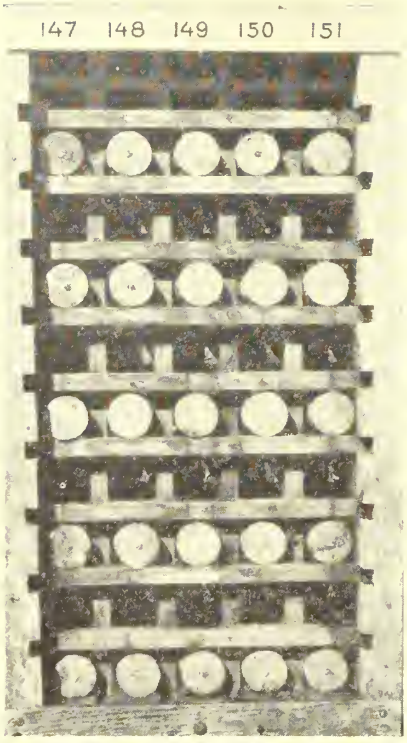


Ottawa sand mortar cylinders made with special cement X in following proportions by volume, after three years in lake: Series 724 1:0.94; series 725—1:1.88; series 726—1:1.88; series 727—1:2.82; series 728—1:4.70

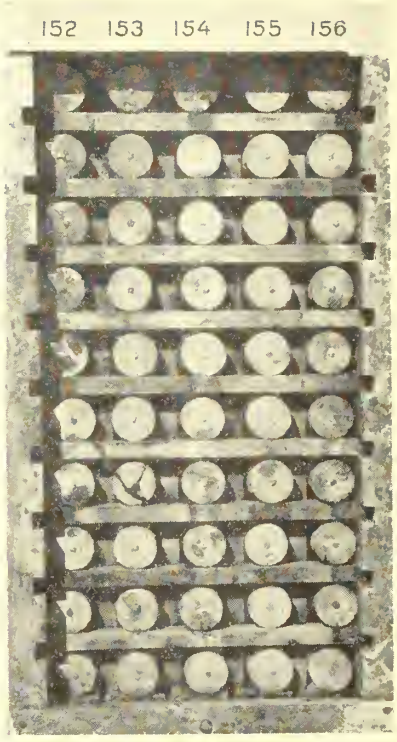
FIGURE 6.—RESULTS OBTAINED WITH CYLINDERS MADE FROM SPECIAL CEMENTS OTHER THAN HIGH ALUMINA



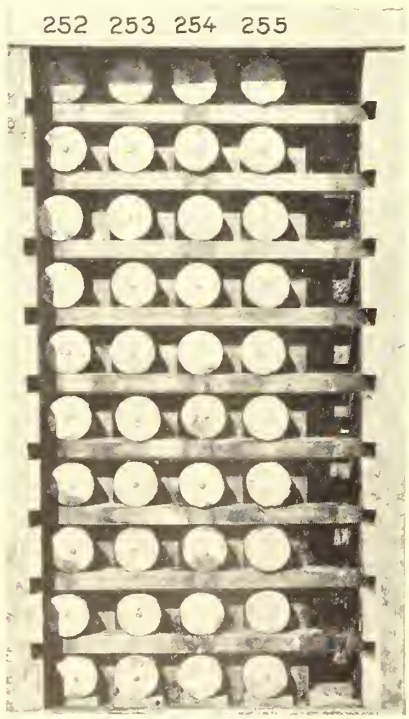
Ottawa sand mortar cylinders made from high alumina cement A, cured as follows, after six years in lake:
Series 142.—Distilled water, 27 days
Series 143.—Moist air, 72 hrs.
Series 144.—Water vapor at 100° F., 48 hrs.
Series 145.—Water vapor at 155° F., 48 hrs.
Series 146.—Steam at 212° F., 48 hrs.



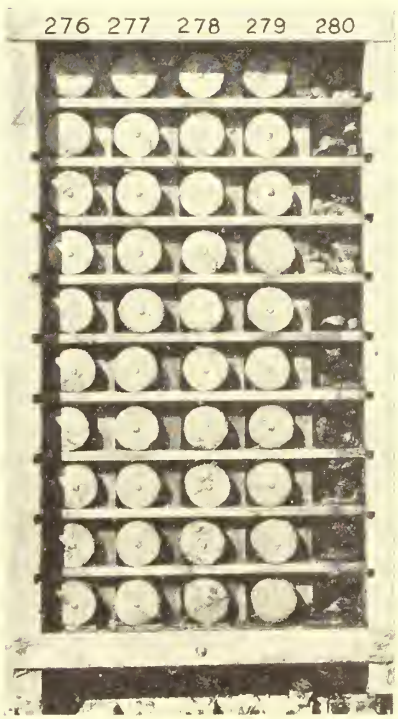
Cylinders made from high alumina cement A, cured as follows, after six years in lake:
Series 147.—Distilled water, 27 days
Series 148.—Moist air, 72 hours
Series 149.—Water vapor at 100° F., 48 hrs.
Series 150.—Water vapor at 155° F., 48 hrs.
Series 151.—Steam at 212° F., 48 hrs.



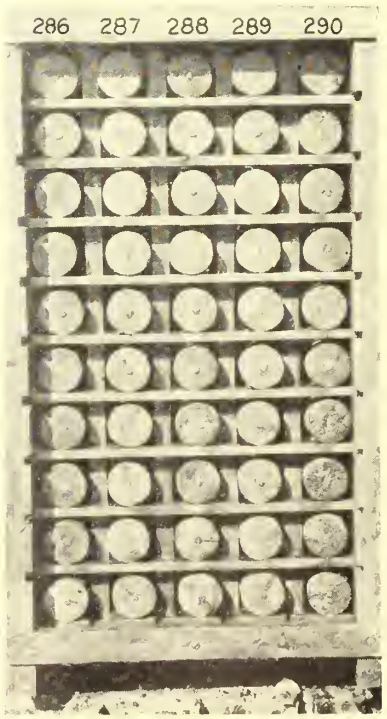
Cylinders made from high alumina cement A, cured as follows, after six years in lake:
Series 152.—Distilled water 20 days, dry air 344 days
Series 153.—Distilled water 20 days, dry air 56 days
Series 154.—Distilled water 20 days, dry air 28 days
Series 155.—Distilled water 20 days, dry air 14 days
Series 156.—Distilled water 20 days



Cylinders made from high alumina cement C, after five and one-half years in lake

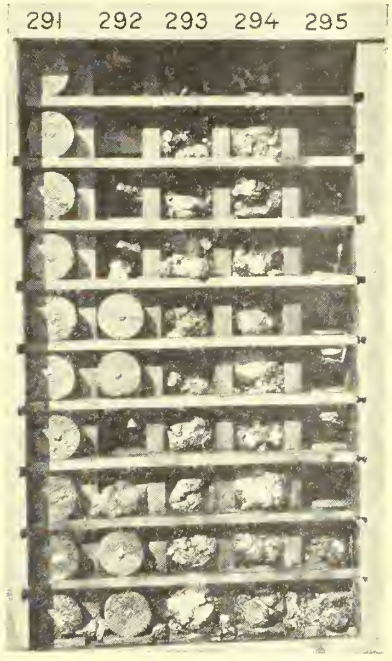


Ottawa sand mortar cylinders made from high alumina cement B in following proportions by volume, after five years in lake: Series 276—1: 2; series 277—1: 3; series 278—1: 3; series 279—1: 4; series 280—1: 5

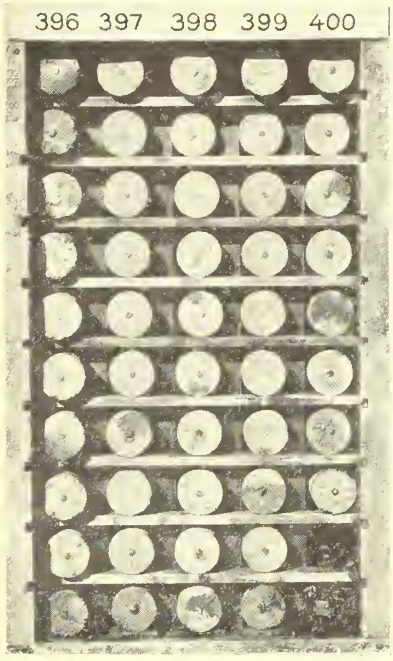


Cylinders made from high alumina cement B in following proportions by volume after five years in lake: Series 286—1: 2; series 287—1: 3; series 288—1: 3; series 289—1: 4; series 290—1: 5

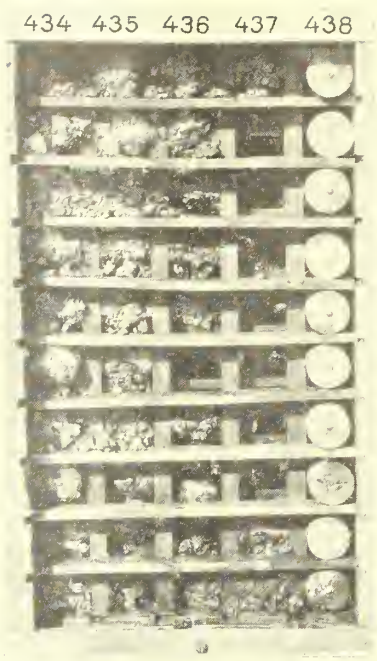
FIGURE 7.—RESULTS OBTAINED WITH CYLINDERS MADE FROM HIGH ALUMINA CEMENTS



Cylinders made up as follows, after five years in lake:
- Series 291.—High alumina C, 1:5 concrete
- Series 292.—High alumina C, 1:5 mortar
- Series 293.—Portland cement mortar
- Series 294.—Portland cement mortar, sulphur impregnated
- Series 295.—Portland cement mortar, 10 per cent sulphur admixed

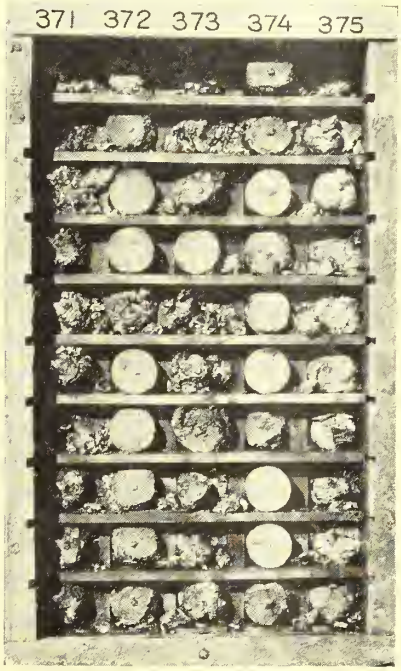


Cylinders made from high alumina cement B with various consistencies, as follows, after five years in lake:
- Series 396.—Relative consistency 0.75, water ratio 0.44
- Series 397.—Relative consistency 0.90 water ratio 0.53
- Series 398.—Relative consistency 1.00, water ratio 0.59
- Series 399.—Relative consistency 1.25, water ratio 0.73
- Series 400.—Relative consistency 1.50, water ratio 0.88

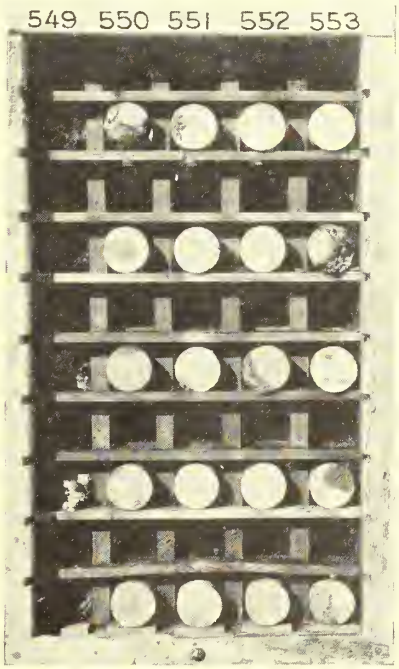


Cylinders using a mixture of Portland cement and high alumina cement B in the following proportions, after four years in lake:
- Series 434.—100 per cent Portland
- Series 435.—5 per cent high alumina B; 95 per cent Portland
- Series 436.—10 per cent high alumina B; 90 per cent Portland
- Series 437.—20 per cent high alumina B; 80 per cent Portland
- Series 438.—100 per cent high alumina B

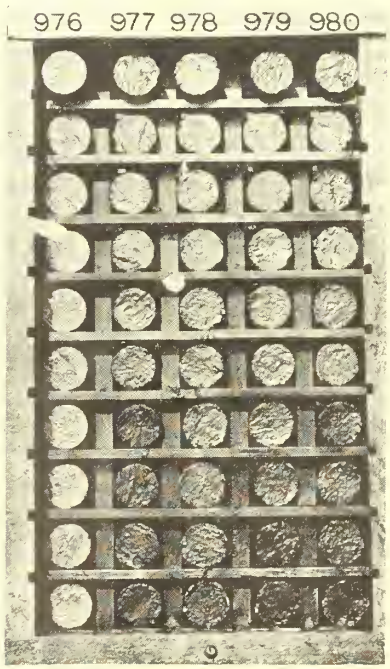
FIGURE 8.—VARIOUS COMPARISONS SHOWING FAVORABLE RESULTS OBTAINED WITH HIGH ALUMINA CEMENTS



Series 371 to 373, concrete cylinders made from standard Portland cement; series 374 and 375, concrete and mortar cylinders, respectively, treated with Inertol. After four and one-half years in lake



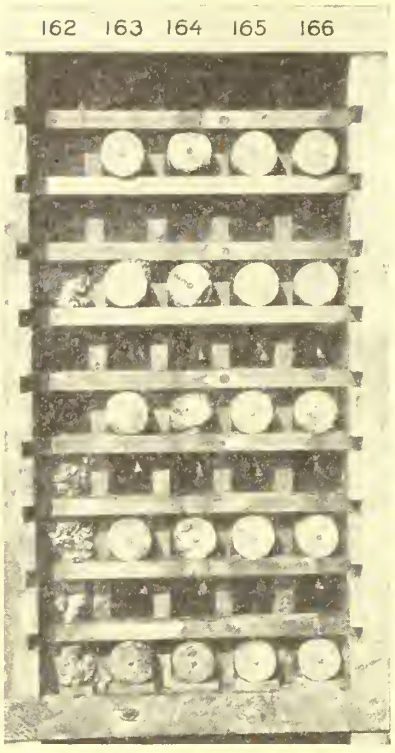
Cylinders surface-treated as follows, after three and one-half years in lake:
- Series 549.—Boiling water ½-minute
- Series 550.—Linseed oil, 70° F., ½-minute (1 coat)
- Series 551.—Linseed oil, 70° F., ½-minute (2 coats)
- Series 552.—Linseed oil, 225° F., ½-minute (1 coat)
- Series 553.—Linseed oil, 225° F., ½-minute (2 coats)



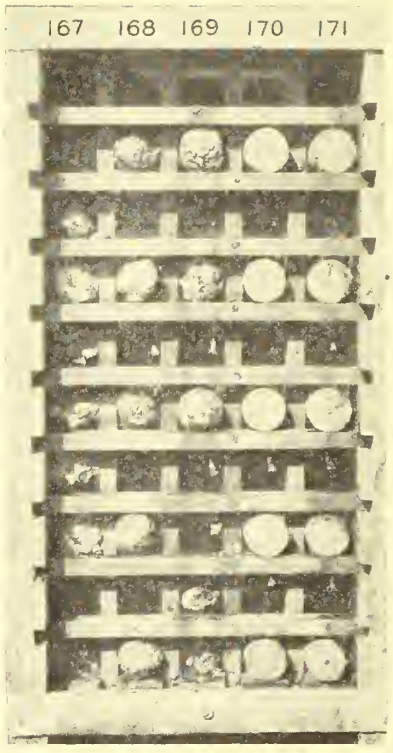
Cylinders with McEverlast protective coatings, after one year in lake

FIGURE 9.—RESULTS OBTAINED FROM SURFACE-TREATED OR IMPREGNATED CYLINDERS

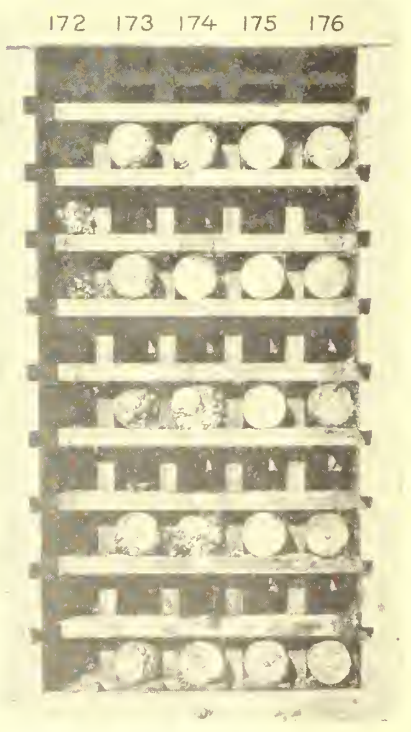
Cylinders with blast furnace slag admixed in following proportions, after five years in lake:
Series 157.—40 per cent
Series 158.—10 per cent
Series 159.—40 per cent
Series 160.—10 per cent (cured in water vapor at 155° F., 48 hrs.)
Series 161.—40 per cent (cured in water vapor at 155° F., 48 hrs.)



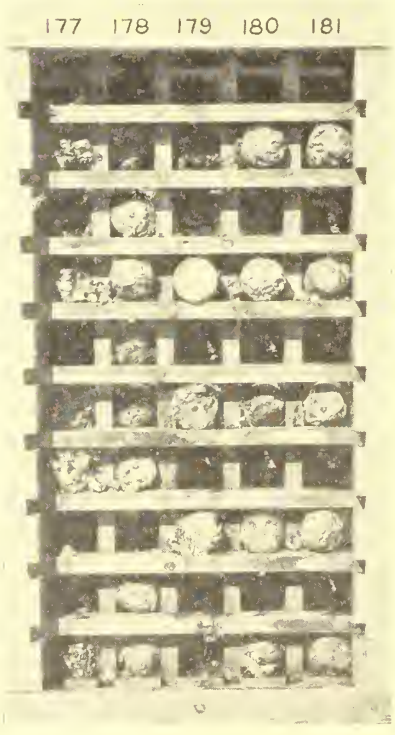
Cylinders with calcium chloride admixed in following proportions, after six years in lake:
Series 162.—4 per cent
Series 163.—8 per cent
Series 164.—4 per cent
Series 165.—8 per cent (cured in water vapor at 155° F., 48 hrs.)
Series 166.—4 per cent (cured in water vapor at 155° F., 48 hrs.)



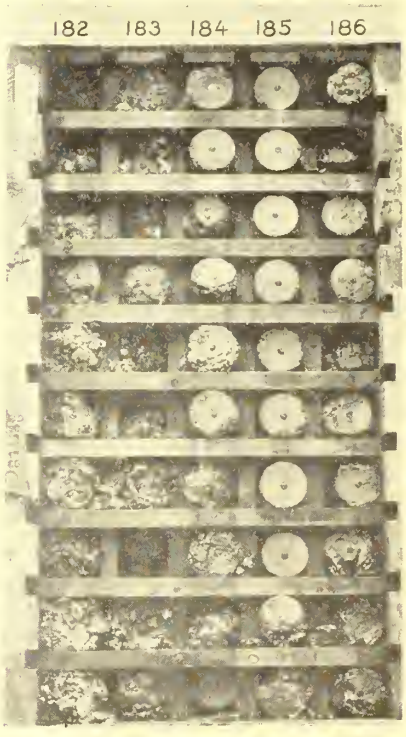
Cylinders with Cal admixed and cured as follows, after six years in lake:
Series 167.—4 per cent
Series 168.—8 per cent
Series 169.—4 per cent
Series 170.—8 per cent (cured in water vapor at 155° F., 48 hrs.)
Series 171.—4 per cent (cured in water vapor at 155° F., 48 hrs.)



Cylinders with Ironite admixed in following proportions, after six years in lake:
Series 172.—No admixture
Series 173.—20 per cent
Series 174.—No admixture (cured in water vapor at 155° F., 48 hrs.)
Series 175.—20 per cent (cured in water vapor at 155° F., 48 hrs.)
Series 176.—20 per cent (cured in water vapor at 100° F., 48 hrs.)



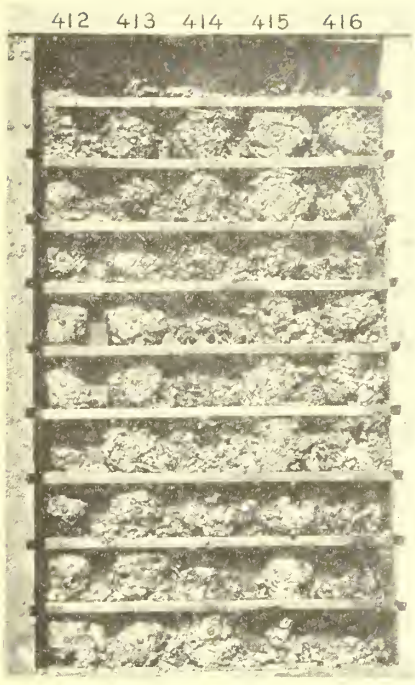
Cylinders with volcanic ash admixedin following proportions, after six years in lake:
Series 177.—No admixture
Series 178.—20 per cent
Series 179.—No admixture (cured in water vapor at 155° F., 48 hrs.)
Series 180.—20 per cent (cured in water vapor at 155° F., 48 hrs.)
Series 181.—20 per cent (cured in water vapor at 100° F., 48 hrs.)



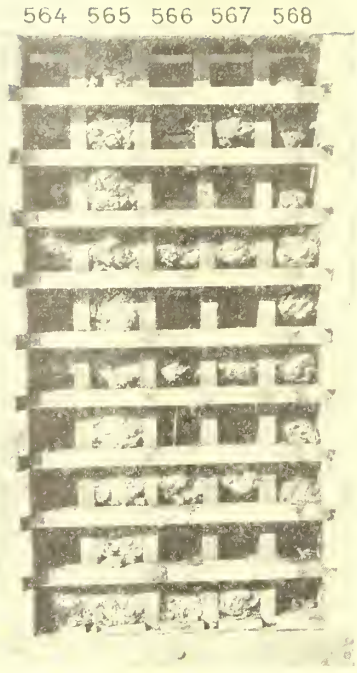
Cylinders with Alkagel "A" admixed in following proportions, after five years in lake:
Series 182.—No admixture
Series 183.—3 per cent
Series 184.—No admixture (cured in water vapor at 155° F., 48 hrs.)
Series 185.—3 per cent (cured in water vapor at 155° F., 48 hrs.)
Series 186.—3 per cent (cured in water vapor at 100° F., 48 hrs.)

FIGURE 10.—RESULTS OBTAINED BY THE USE OF VARIOUS ADMIXTURES

Cylinders with Trass admixed as follows, after four and one-half years in lake:
Series 391.—No admixture
Series 392.—33 per cent
Series 393.—66 per cent
Series 394.—33 per cent
Series 395.—66 per cent

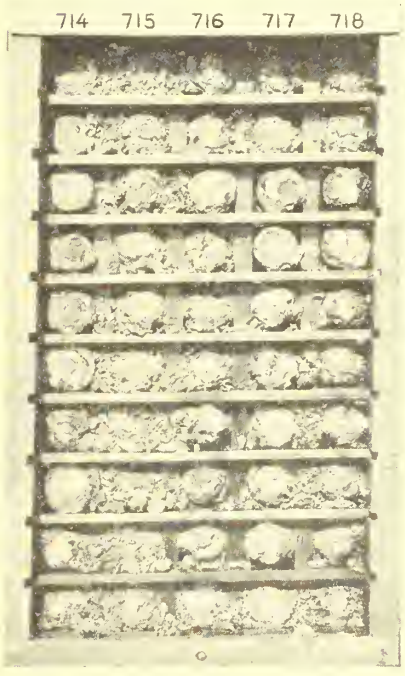


Cylinders of Series 412, 2½ per cent Celite admixed; cylinders of Series 413 to 416 made from different brands of standard Portland cement. After three years in lake

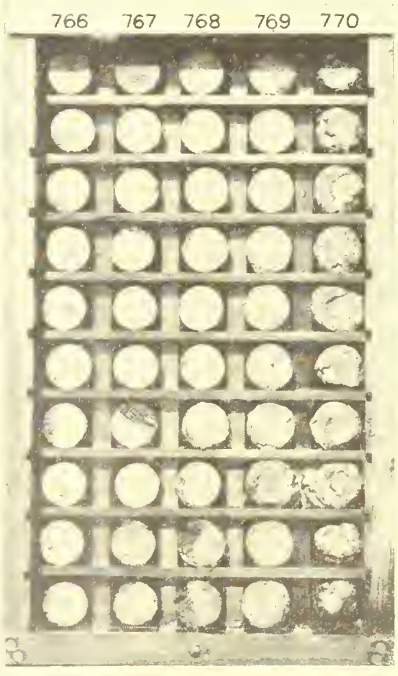


Cylinders with powdered fuel ash admixed as follows, after three years in lake:
Series 564 and 565.—2.5 per cent
Series 566.—5 per cent
Series 567 and 568.—10 per cent

Cylinders with Truscon and $BaCl_2$ admixed as follows, after three years in lake:
Series 578.—1 per cent Truscon
Series 579.—2 per cent Truscon
Series 580.—4 per cent Truscon
Series 581.—6 per cent $BaCl_2$
Series 582.—12 per cent $BaCl_2$

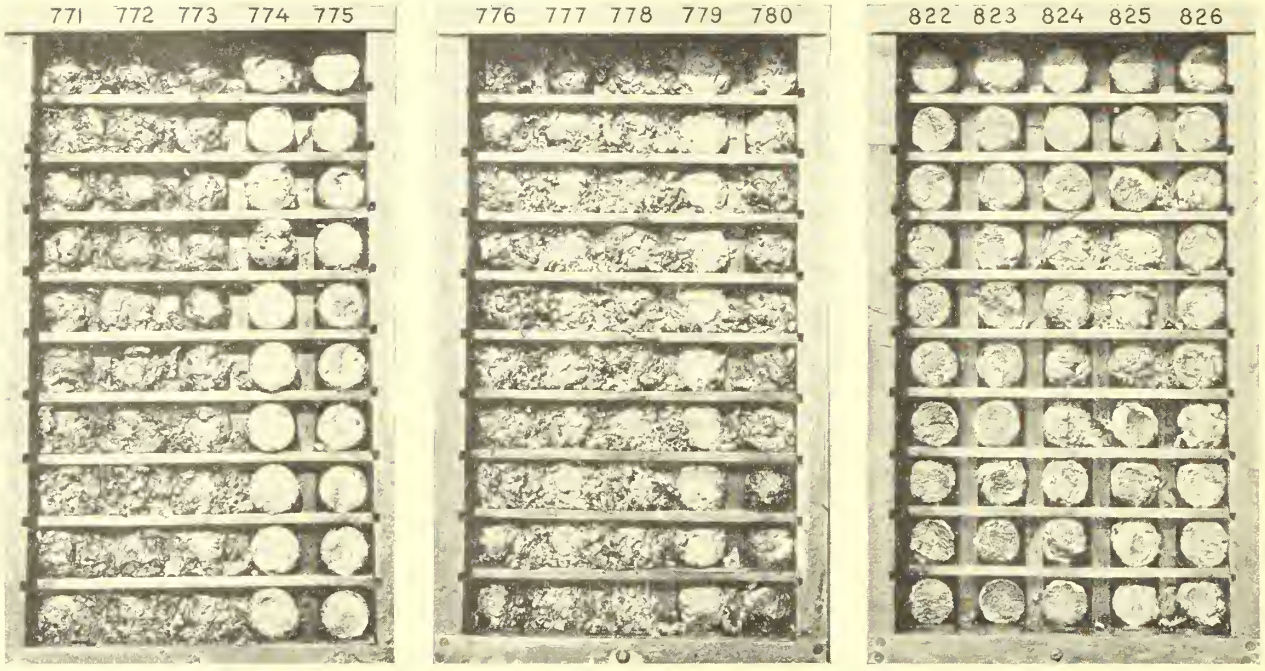


Cylinders made from special cement D and cylinders with Earthcrete admixed, after three years in lake:
Series 714.—Special cement D, distilled water 20 days, air 35 days
Series 715.—Portland cement, distilled water 20 days, air 35 days
Series 716.—Portland cement, 0.27 per cent Earthcrete, distilled water 20 days, air 35 days
Series 717.—Portland cement, 1.06 per cent Earthcrete, distilled water 20 days, air 35 days
Series 718.—Portland cement, 0.27 per cent Earthcrete, water vapor at 155° F., 48 hrs., air 53 days



Cylinders of Series 767 and 769 were made from standard Portland cement. Series 766 and 768 were made from special cements from the same respective mills. Series 770 contained a 2 per cent admixture of Medusa waterproofing. After one and one-half years in lake

FIGURE 11.—RESULTS OBTAINED BY THE USE OF VARIOUS ADMIXTURES



Cylinders with Omicron, admixed as follows, after one and one-half years in lake:
Series 771.—No admixture
Series 772.—3.75 per cent
Series 773.—7.5 per cent
Series 774.—15 per cent
Series 775.—30 per cent

Cylinders with Colloy and Medusa waterproofing admixed as follows, after one and one-half years in lake:
Series 776.—No admixture
Series 777.—2 per cent Colloy
Series 778.—4 per cent Colloy
Series 779.—1 per cent Medusa waterproofing
Series 780.—4 per cent Medusa waterproofing

Cylinders with Barnsdall admixed as follows, after one year in lake:
Series 822.—No admixture
Series 823.—3.75 per cent
Series 824.—7.5 per cent
Series 825.—15 per cent
Series 826.—30 per cent

FIGURE 12.—RESULTS OBTAINED BY THE USE OF VARIOUS ADMIXTURES

TABLE 2.—*Record of tests of 2 by 4 inch concrete cylinders exposed to the action of sulphate water of Medicine Lake, S. Dak., and parallel tests of cylinders stored in tap water in laboratory tanks*

[Unless otherwise noted the fineness modulus of aggregate is 4.67 and the mix is 1:3. Each test result, with a few exceptions, is an average of five cylinders made on different days]

PART 1.—CURING IN WATER VAPOR OR STEAM BETWEEN TEMPERATURES OF 100° AND 365° F.

Series No.	Cement laboratory No.	Cement	Water ratio	Curing method: Time in moist closet	Time in water	Time in water vapor or steam	Temperature of water vapor or steam	Time in air	Absorption at 21 days	Average of compression tests: Tank specimens: 7 days	28 days	1 year	5 years	Lake specimens: 1 year	Percentage of normal strength as indicated by tank specimens at 1 year	3 years	5 years	Percentage of normal strength as indicated by tank specimens at 5 years
				Hours	*Days*	*Hours*	*°F.*	*Days*	*Per cent*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*		*Lbs. per sq. in.*	*Lbs. per sq. in.*	
87	11	½ Portland A, ½ Portland B	0.59	3	------	69	100	25	6.6	3,730	4,120	5,110	5,470	1,430	28	0	0	----------
88	11	------do	.59	6	------	66	100	25	6.7	3,590	4,320	4,960	5,450	1,370	28	0	0	----------
89	11	------do	.59	12	------	60	100	25	6.7	4,050	4,680	5,270	5,200	1,290	25	0	0	----------
90	11	------do	.59	24	------	48	100	25	6.7	4,080	4,220	5,240	6,270	1,630	31	0	0	----------
91	11	------do	.59	48	------	24	100	25	6.8	3,510	4,060	5,010	4,970	1,920	38	0	0	----------
92	11	------do	.59	24	27	------	------	28	6.7	2,620	4,540	5,690	5,400	3,780	66	0	0	----------
93	11	------do	.59	72	------	------	------	25	7.0	3,380	3,560	5,120	5,140	3,150	62	0	0	----------
94	11	------do	.59	24	------	48	100	25	6.8	3,810	4,000	5,640	5,910	2,020	36	0	0	----------
313	65	M	.60	24	20	------	------	35	5.9	3,150	3,950	6,050	5,680	2,250	37	0	0	----------
315	65	------do	.60	24	------	48	100	53	6.2	3,740	3,800	5,970	6,320	4,230	71	3,390	3,330	53
[1] 112	17	½ Portland A, ½ Portland B	.64	24	27	------	------	------	10.0	1,650	2,630	3,540	2,790	1,790	51	0	0	----------
[1] 113	17	------do	.64	72	------	------	------	25	10.7	2,000	2,000	2,980	2,940	840	28	0	0	----------
[1] 114	17	------do	.64	24	------	48	100	25	10.1	2,270	2,790	3,040	3,050	630	21	0	0	----------
82	11	------do	.59	3	------	69	155	25	6.9	3,360	4,210	5,380	5,550	500	9	0	0	----------
83	11	------do	.59	6	------	66	155	25	6.9	3,840	4,720	5,700	5,620	490	9	0	0	----------
84	11	------do	.59	12	------	60	155	25	6.7	4,440	4,960	5,250	5,920	1,030	20	0	0	----------
85	11	------do	.59	24	------	48	155	25	6.6	4,350	4,550	6,460	5,830	1,220	19	0	0	----------
86	11	------do	.59	48	------	24	155	25	6.7	3,540	4,210	6,250	6,320	1,660	27	0	0	----------
92	11	------do	.59	24	27	------	------	28	6.7	2,620	4,540	5,690	5,400	3,780	66	0	0	----------
93	11	------do	.59	72	------	------	------	25	7.0	3,380	3,560	5,120	5,140	3,150	62	0	0	----------
95	11	------do	.59	24	------	48	155	25	6.4	4,240	4,370	5,510	5,510	1,660	30	0	0	----------
97	11, 8	------do	.59	24	------	48	155	25	6.3	3,900	4,770	5,690	5,970	1,880	33	0	0	----------
99	11, 8	------do	.59	24	------	72	155	24	6.4	4,490	4,830	5,410	5,990	1,530	28	0	0	----------
102	17	------do	.59	24	------	48	155	25	6.2	4,550	4,730	5,580	5,960	980	18	0	0	----------
104	17	------do	.59	24	------	72	155	24	6.1	4,910	5,180	5,180	5,630	1,270	25	0	0	----------
174	18	------do	.59	24	------	48	155	35	6.1	4,020	4,390	4,330	5,650	3,910	90	----------	2,740	48
179	18	------do	.59	24	------	48	155	35	5.8	4,110	4,510	5,210	6,460	3,820	73	----------	1,750	27

[1] Standard Ottawa sand cylinders.

TABLE 2.—*Record of tests of 2 by 4 inch concrete cylinders exposed to the action of sulphate water of Medicine Lake, S. Dak., and parallel tests of cylinders stored in tap water in laboratory tanks*—Continued

PART 1.—CURING IN WATER VAPOR OR STEAM BETWEEN TEMPERATURES OF 100° AND 365° F.—Continued

Series No.	Cement laboratory No.	Cement	Curing method						Average of compression tests								
									Tank specimens				Lake specimens				
			Water ratio	Time in moist closet	Time in water	Time in water vapor or steam	Temperature of water vapor or steam	Time in air	Absorption at 21 days	7 days	28 days	1 year	5 years	1 year	Percentage of normal strength as indicated by tank specimens at 1 year	3 years	5 years	Percentage of normal strength as indicated by tank specimens at 5 years
				Hours	*Days*	*Hours*	*°F.*	*Days*	*Per cent*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*		*Lbs. per sq. in.*	*Lbs. per sq. in.*	
184	18	½ Portland A, ½ Portland B	0.59	24	------	48	155	35	6.0	4,070	4,530	5,100	6,110	4,240	83	-------	800	13
313	65	M	.60	24	20	-------	-------	35	5.9	3,150	3,950	6,050	5,680	2,250	37	0	0	-------
314	65	do	.60	24	------	48	155	53	6.0	4,460	4,010	6,170	6,810	5,430	88	3,920	3,690	54
[1] 107	17	½ Portland A, ½ Portland B	.64	24	------	48	155	25	9.8	2,660	3,080	3,580	3,580	770	22	0	0	-------
[1] 109	17	do	.64	24	------	72	155	24	9.6	2,670	3,220	3,490	3,580	930	27	0	0	-------
[1] 112	17	do	.64	24	27	-------	-------	-------	10.0	1,650	2,630	3,540	2,790	1,790	51	0	0	-------
[1] 113	17	do	.64	72	------	-------	-------	25	10.7	2,000	2,000	2,980	2,940	840	28	0	0	-------
[1] 115	17	do	.64	24	------	48	155	25	10.1	2,730	2,900	3,400	3,020	860	25	0	0	-------
[1] 127	17	do	.64	24	------	48	155	53	9.1	2,530	2,710	3,460	3,260	2,280	66	-------	0	-------
[1] 129	17	do	.64	24	------	72	155	52	9.0	2,700	3,120	3,320	3,230	1,970	59	-------	0	-------
351	74	do	.62	24	------	12	190	54	6.6	2,650	2,940	5,320	6,870	1,730	33	0	0	-------
391	74	do	.62	24	20	-------	-------	35	6.3	3,130	4,890	6,060	6,430	2,480	41	0	0	-------
77	11	do	.59	3	------	69	212	25	7.9	1,110	1,250	1,250	2,810	1,210	97	1,340	2,630	94
78	11	do	.59	6	------	66	212	25	7.2	2,260	3,040	3,130	5,900	2,840	91	2,930	4,120	70
79	11	do	.59	12	------	60	212	25	6.9	3,400	4,120	4,550	6,020	4,410	97	4,940	5,400	90
80	11	do	.59	24	------	48	212	25	6.9	3,060	4,020	4,600	5,790	4,330	94	4,860	5,150	89
81	11	do	.59	48	------	24	212	25	6.7	2,910	3,850	4,630	6,730	4,410	95	5,340	6,000	89
92	11	do	.59	24	27	-------	-------	28	6.7	2,620	4,540	5,690	5,400	3,780	66	0	0	-------
96	11	do	.59	24	------	48	212	25	6.5	3,770	4,410	4,560	5,720	4,300	94	-------	4,930	86
98	11, 8	do	.59	24	------	24 24	155 212	25	6.7	4,180	4,120	5,020	6,190	4,340	87	-------	5,310	86
100	11, 8	do	.59	24	------	48 24	155 212	24	6.2	4,310	4,960	5,730	5,750	5,540	97	-------	5,890	102
101	11, 8	do	.59	24	------	24 48	155 212	24	6.3	4,110	4,710	4,890	6,490	4,640	95	-------	5,330	82
103	17	do	.59	24	------	24 24	155 212	25	6.0	4,600	5,510	5,290	6,280	4,980	94	5,510	4,910	78
105	17	do	.59	24	------	48 24	155 212	24	6.0	4,820	5,070	4,910	6,720	5,220	106	6,000	5,680	85
106	17	do	.59	24	------	24 48	155 212	24	5.7	5,110	5,900	5,320	6,020	5,250	99	5,830	5,720	95
296	61	do	.62	24	------	48	212	53	6.5	3,330	3,660	5,020	5,760	4,390	88	5,840	5,950	103
297	63	Portland A	.60	24	------	48	212	53	6.4	3,650	3,660	4,680	6,020	4,600	98	4,960	5,480	91
298	62	Portland B	.62	24	------	48	212	53	6.9	3,700	3,830	4,580	6,440	4,480	98	4,840	5,600	87
299	33	Portland D	.60	24	------	48	212	53	6.4	3,690	3,770	4,620	6,330	4,270	92	4,910	5,900	93
300	34	Portland C	.67	24	------	48	212	53	6.6	3,880	4,540	4,910	6,110	4,890	99	5,690	5,810	95
301	61	½ Portland A, ½ Portland B	.62	24	------	48	212	53	6.5	3,690	3,610	4,500	4,840	4,480	100	5,160	5,300	110
302	35	Portland E	.60	24	------	48	212	53	6.2	3,540	3,530	4,550	5,040	4,450	98	4,660	4,640	92
303	37	Portland F	.60	24	------	48	212	53	6.0	3,550	3,900	5,060	4,890	4,920	97	5,920	5,470	112
304	39	Portland G	.60	24	------	48	212	53	6.2	3,740	3,920	4,990	5,870	4,680	94	5,060	5,060	86
305	41	Portland H	.60	24	------	48	212	53	6.1	3,730	3,840	4,860	5,380	4,410	91	4,340	5,100	95
306	61	½ Portland A, ½ Portland B	.62	24	------	48	212	53	6.2	3,280	3,920	4,480	6,570	4,410	98	5,990	5,230	80
307	40	Portland I	.60	24	------	48	212	53	5.9	3,420	4,150	5,090	5,850	4,190	82	4,990	4,530	77
308	42	Portland J	.62	24	------	48	212	53	6.4	2,650	3,240	4,680	5,310	3,480	74	4,440	4,580	86
309	55	Portland K1	.62	24	------	48	212	53	6.5	3,260	4,200	4,950	5,800	4,910	99	4,910	4,960	86
310	60	Portland L	.60	24	------	48	212	53	6.2	3,260	3,870	5,060	6,380	4,460	88	4,720	5,770	90
311	65	Portland M	.60	24	------	48	212	53	5.5	4,660	5,580	5,810	6,740	5,570	96	5,420	5,960	88
313	65	do	.60	24	20	-------	-------	35	5.9	3,150	3,950	6,050	5,680	2,250	37	0	0	-------
352	74	½ Portland A, ½ Portland B	.62	24	------	12	212	54	6.9	2,450	2,910	5,230	6,070	4,240	81	3,850	0	-------
391	74	do	.62	24	20	-------	-------	35	6.3	3,130	4,890	6,060	6,430	2,480	41	0	0	-------
584	139	do	.62	24	20	1 min.	212	35	6.3	3,740	5,150	5,790	-------	3,350	58	0	0	-------
585	139	do	.62	24	------	1 min.	212	55	6.4	2,770	2,860	5,430	-------	5,370	99	0	0	-------
586–587	139	do	.62	24	------	-------	-------	1	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------
635–636	139	do	.64	24	------	1 min. 48	212	54 53	6.6 6.3	2,970 3,960	2,920 4,330	5,800 4,250	-------	4,460 4,370	76 103	0 5,950	0	-------
640	139	do	.64	24	20	-------	-------	35	6.5	3,320	4,470	5,800	-------	4,530	78	0	0	-------
641	139	do	.64	24	------	48	212	53	6.9	3,590	3,760	4,530	-------	3,740	83	5,360	-------	-------
830	232	do	.62	24	20	-------	-------	35	5.9	3,230	4,720	6,360	-------	3,950	62	-------	-------	-------
831	232	do	.62	24	------	¾	212	55	6.9	3,020	2,880	5,740	-------	4,210	73	-------	-------	-------
832	232	do	.62	24	------	1½	212	55	7.4	2,590	2,400	5,890	-------	4,610	78	-------	-------	-------
833	232	do	.62	24	------	3	212	55	7.2	2,890	3,130	6,350	-------	4,510	71	-------	-------	-------
834	232	do	.62	24	------	6	212	55	7.0	3,220	3,470	5,730	-------	4,970	87	-------	-------	-------
835	232	do	.62	24	------	12	212	55	7.1	3,570	3,580	5,070	-------	5,250	104	-------	-------	-------
836	232	do	.62	24	------	24	212	54	7.2	3,960	4,190	5,750	-------	5,770	100	-------	-------	-------
837	232	do	.62	24	------	48	212	53	6.6	4,360	4,670	4,860	-------	4,620	95	-------	-------	-------
838	232	do	.62	24	------	96	212	51	6.1	4,330	4,990	5,030	-------	4,210	84	-------	-------	-------
839	232	do	.62	24	------	192	212	47	6.1	5,180	6,050	5,130	-------	4,550	89	-------	-------	-------
[1] 108	17	do	.64	24	------	24 24	155 212	25	9.8	2,590	2,910	2,930	2,880	2,540	87	3,390	2,580	90
[1] 110	17	do	.64	24	------	48 24	155 212	24	9.5	2,600	3,160	2,940	2,590	2,720	93	3,160	2,360	91
[1] 111	17	do	.64	24	------	24 48	155 212	24	9.7	2,510	3,130	2,630	3,020	2,450	93	2,820	2,320	77
[1] 112	17	do	.64	24	27	-------	-------	-------	10.0	1,650	2,630	3,540	2,790	1,790	51	0	0	-------
[1] 113	17	do	.64	72	------	-------	-------	25	10.7	2,000	2,000	2,980	2,940	840	28	0	0	-------
[1] 116	17	do	.64	24	------	48	212	25	10.3	2,220	2,470	2,500	3,250	2,430	97	-------	2,460	76

[1] Standard Ottawa sand cylinders.

TABLE 2.—*Record of tests of 2 by 4 inch concrete cylinders exposed to the action of sulphate water of Medicine Lake, S. Dak., and parallel tests of cylinders stored in tap water in laboratory tanks*—Continued

PART 1.—CURING IN WATER VAPOR OR STEAM BETWEEN TEMPERATURES OF 100° AND 365° F.—Continued

Series No.	Cement laboratory No.	Cement	Water ratio	Curing method					Absorption at 21 days	Average of compression tests								
										Tank specimens				Lake specimens				
				Time in moist closet	Time in water	Time in water vapor or steam	Temperature of water vapor or steam	Time in air		7 days	28 days	1 year	5 years	1 year	Percentage of normal strength as indicated by tank specimens at 1 year	3 years	5 years	Percentage of normal strength as indicated by tank specimens at 5 years
				Hours	*Days*	*Hours*	*°F.*	*Days*	*Per cent*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*		*Lbs. per sq. in.*	*Lbs. per sq. in.*	
[1] 128	17	½ Portland A, ½ Portland B	0.64	24		24	155	53	9.1	2,350	2,480	2,830	3,340	3,180	112		2,040	61
						24	212											
[1] 130	17	do	.64	24		48	155	52	9.1	2,750	2,720	2,830	3,130	3,080	109		1,740	56
						24	212											
[1] 131	17	do	.64	24		24	155	52	9.2	2,630	2,470	2,600	2,940	2,560	98		1,730	59
						48	212											
[2] 353	74	do	.62	24		12	235	54	7.3	1,900	2,750	4,780	5,410	3,800	79	3,970	2,480	46
391	74	do	.62	24	20			35	6.3	3,130	4,890	6,060	6,430	2,480	41	0	0	
811	219	do	.62	24	20			35	5.5	3,620	4,930	6,350		0		0	0	
812	219	do	.62	24		¾	230	55	7.0	2,060	2,170	5,530		5,160	93			
813	219	do	.62	24		1½	230	55	6.9	1,830	2,190	5,620		4,530	81			
814	219	do	.62	24		3	230	55	6.7	2,180	2,320	6,020		4,860	81			
815	219	do	.62	24		6	230	55	6.6	2,390	2,990	5,520		5,320	96			
820	222	do	.62	24		12	230	103	6.8	2,640	3,200	5,260		4,990	95			
821	222	do	.62	24		24	230	102	6.6	3,560	3,250	4,740		4,520	95			
829	225	do	.62	24		48	230	87	6.9	3,900	4,650	4,640		4,360	94			
855	232	do	.62	24	20			35	5.7	3,840	5,310	5,540		4,070	73			
869	232	do	.62	24		96	230	46	6.1	4,480	4,600	4,480		4,600	103			
870	232	do	.62	24		192	230	42	6.2	3,830	4,450	3,360		3,560	106			
[3] 354	74	do	.62	24		12	260	54	7.1	2,270	2,490	4,880	5,570	3,760	77	4,160	3,320	60
391	74	do	.62	24	20			35	6.3	3,130	4,890	6,060	6,430	2,480	41	0	0	
806	219	do	.62	24	20			35	5.8	3,450	4,620	6,110		1,620	27			
807	219	do	.62	24		¾	260	55	7.4	1,960	1,930	5,400		4,610	85			
808	219	do	.62	24		1½	260	55	7.1	2,000	2,130	5,490		4,990	91			
809	219	do	.62	24		3	260	55	7.2	2,170	2,460	5,090		5,200	102			
810	219	do	.62	24		6	260	55	7.0	2,790	2,870	5,380		4,940	92			
818	222	do	.62	24		12	260	97	6.7	3,190	3,380	5,560		4,630	83			
819	222	do	.62	24		24	260	95	6.7	3,480	3,810	4,620		4,520	98			
828	225	do	.62	24		48	260	83	6.6	3,720	3,880	3,990		3,790	95			
855	232	do	.62	24	20			35	5.7	3,840	5,310	5,540		4,070	73			
867	232	do	.62	24		96	260	48	6.4	3,820	3,810	3,460		3,550	103			
868	232	do	.62	24		192	260	44	6.6	3,260	3,690	3,420		2,940	86			
[4] 355	74	do	.62	24		12	285	54	7.2	2,260	2,870	4,340	5,250	3,870	89	3,970	4,190	80
391	74	do	.62	24	20			35	6.3	3,130	4,890	6,060	6,430	2,480	41	0	0	
430	97, 98	Portland K1	.64	24		12	285	54	6.6	2,960	3,470	3,950	4,830	3,680	93	5,000	4,210	87
431	86, 99	Portland I	.64	24		12	285	54	6.1	3,650	4,150	4,470	5,330	4,290	96	5,440	5,440	102
432	82, 95	Portland C	.67	24		12	285	54	6.9	3,330	3,820	5,530	6,550	4,930	89	5,460	5,410	83
433	83	Portland H	.64	24		12	285	54	6.5	3,460	3,950	5,110	6,520	4,840	95	5,510	5,980	92
801	219	½ Portland A, ½ Portland B	.62	24	20			35	5.8	3,740	6,260	5,790		2,820	49			
802	219	do	.62	24		¾	285	55	7.3	1,780	1,740	5,240		5,570	106			
803	219	do	.62	24		1½	285	55	7.2	2,010	2,220	6,550		5,560	85			
804	219	do	.62	24		3	285	55	7.3	2,420	2,590	5,560		5,650	102			
805	219	do	.62	24		6	285	55	6.8	3,080	3,370	5,860		5,600	96			
816	222	do	.62	24		12	285	87	6.5	3,320	3,420	4,940		4,510	91			
817	222	do	.62	24		24	285	85	6.3	3,740	4,160	4,540		4,750	105			
827	225	do	.62	24		48	285	64	6.7	3,300	3,600	3,200		3,170	99			
855	232	do	.62	24	20			35	5.7	3,840	5,310	5,540		4,070	73			
865	232	do	.62	24		96	285	45	6.5	3,550	4,230	3,550		4,190	118			
866	232	do	.62	24		192	285	41	6.5	3,540	4,300	4,270		4,530	106			
[5] 840	232	do	.62	24	20			35	5.8	3,810	5,050	6,010		3,060	51			
841	232	do	.62	24		¾	315	55	6.9	2,010	2,020	5,680		5,130	90			
842	232	do	.62	24		1½	315	55	6.8	2,490	2,640	5,550		5,390	97			
843	232	do	.62	24		3	315	55	6.5	3,090	3,030	5,620		5,220	93			
844	232	do	.62	24		6	315	55	6.8	3,560	3,900	5,110		5,100	100			
845	232	do	.62	24		12	315	55	6.5	3,410	3,780	4,150		4,340	105			
846	232	do	.62	24		24	315	54	6.8	3,180	3,750	3,440		2,770	81			
847	232	do	.62	24		48	315	53	6.8	3,440	3,970	3,160		3,240	103			
848	232	do	.62	24		96	315	51	6.6	3,430	4,280	3,090		3,620	117			
849	232	do	.62	24		192	315	47	6.8	3,580	4,640	3,130		4,470	143			
[6] 855	232	do	.62	24	20			35	5.7	3,840	5,310	5,540		4,070	73			
856	232	do	.62	24		¾	350	55	7.3	1,670	1,940	5,710		4,790	84			
857	232	do	.62	24		1½	350	55	7.0	2,500	2,800	5,950		5,350	90			
858	232	do	.62	24		3	350	55	6.7	3,210	3,310	5,610		5,170	92			
859	232	do	.62	24		6	350	55	6.8	3,110	3,540	4,300		4,130	96			
860	232	do	.62	24		12	350	55	6.7	3,280	3,470	3,880		3,760	97			
861	232	do	.62	24		24	350	54	6.8	3,830	3,840	4,700		4,220	90			
862	232	do	.62	24		48	350	53	6.5	3,530	4,710	4,580		4,800	105			
863	232	do	.62	24		96	350	51	6.3	4,140	4,320	4,670		4,530	97			
864	232	do	.62	24		192	350	47	6.2	3,770	4,260	5,000		4,310	86			

[1] Standard Ottawa sand cylinders.
[2] For all series cured in steam at 230° or 235° F. the gauge pressure was 6.1 lbs. per sq. in.
[3] For all series cured in steam at 260° F. the gauge pressure was 20.7 lbs. per sq. in.
[4] For all series cured in steam at 285° F. the gauge pressure was 38.5 lbs. per sq. in.
[5] For all series cured in steam at 315° F. the gauge pressure was 68.7 lbs. per sq. in.
[6] For all series cured in steam at 350° F. the gauge pressure was 119.8 lbs. per sq. in.

TABLE 2.—*Record of tests of 2 by 4 inch concrete cylinders exposed to the action of sulphate water of Medicine Lake, S. Dak., and parallel tests of cylinders stored in tap water in laboratory tanks*—Continued

PART 2.—PORTLAND CEMENTS FROM DIFFERENT MILLS

Series No.	Cement laboratory No.	Cement	Water ratio	Curing method			Absorption at 21 days	Average of compression tests								
								Tank specimens				Lake specimens				
				Time in moist closet	Time in water	Time in air		7 days	28 days	1 year	5 years	1 year	Percentage of normal strength as indicated by tank specimens at 1 year	3 years	5 years	Percentage of normal strength as indicated by tank specimens at 5 years
				Hours	*Days*	*Days*	*Per cent*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*		*Lbs. per sq. in.*	*Lbs. per sq. in.*	
172	18	½ Portland A, ½ Portland B	0.59	24	20	35	5.6	2,870	4,190	5,300	5,400	4,370	82	-------	0	-------
177	18	do	.59	24	20	35	5.9	2,520	4,200	4,930	6,120	3,670	74	-------	0	-------
256	61	do	.59	24	20	35	5.8	2,780	3,950	5,200	6,520	1,480	28	0	0	-------
391	74	do	.62	24	20	35	6.3	3,130	4,890	6,060	6,430	2,480	41	0	0	-------
434	74	do	.66	24	20	35	6.4	2,590	4,030	5,050	5,830	4,870	96	0	0	-------
554	129	do	.64	24	20	35	6.3	3,570	4,760	5,980	-------	3,530	59	0	0	-------
583	139	do	.62	24	20	35	6.3	3,810	4,950	6,420	-------	3,650	57	0	0	-------
640	139	do	.64	24	20	35	6.5	3,320	4,470	5,800	-------	4,530	78	0	0	-------
715	176	do	.62	24	20	35	5.6	3,830	5,100	6,420	-------	4,550	71	0	0	-------
771	204	do	.62	24	20	35	5.9	3,450	4,440	5,870	-------	0	-------	0	0	-------
776	204	do	.62	24	20	35	6.2	3,400	4,420	6,480	-------	0	-------	0	0	-------
801	219	do	.62	24	20	35	5.8	3,740	6,260	5,790	-------	2,820	49	-------	-------	-------
806	219	do	.62	24	20	35	5.8	3,450	4,620	6,110	-------	1,620	27	-------	-------	-------
811	219	do	.62	24	20	35	5.5	3,620	4,930	6,350	-------	0	-------	0	0	-------
822	225	do	.62	24	20	35	5.3	3,220	5,230	6,250	-------	3,800	61	-------	-------	-------
830	232	do	.62	24	20	35	5.9	3,230	4,720	6,360	-------	3,950	62	-------	-------	-------
840	232	do	.62	24	20	35	5.8	3,810	5,050	6,010	-------	3,060	51	-------	-------	-------
855	232	do	.62	24	20	35	5.7	3,840	5,310	5,540	-------	4,070	73	-------	-------	-------
976	237	do	.62	24	20	35	6.0	3,700	4,590	5,750	-------	4,650	81	-------	-------	-------
258	62	Portland B1	.59	24	20	35	5.8	2,960	3,790	4,580	5,780	800	17	0	0	-------
655	156	do	.62	24	20	35	6.4	3,670	5,070	5,800	-------	2,870	50	0	0	-------
665	156	do	.62	24	20	-------	5.9	3,500	5,020	5,820	-------	3,860	66	0	0	-------
670	161	do	.62	24	20	-------	6.2	3,550	5,470	7,100	-------	3,050	43	0	0	-------
675	161	do	.62	24	20	35	5.8	3,490	5,320	6,450	-------	3,870	60	0	0	-------
690	166	do	.62	24	20	-------	6.4	2,830	4,580	6,150	-------	3,810	62	0	0	-------
695	166	do	.62	24	20	35	6.8	2,960	4,340	6,510	-------	4,210	65	0	0	-------
699	170	do	.62	24	20	35	5.8	3,660	5,580	6,250	-------	3,130	50	0	0	-------
709	170	do	.62	24	20	-------	5.8	4,020	4,910	6,940	-------	2,930	42	0	0	-------
730	178	do	.62	24	20	-------	5.6	3,400	4,610	5,780	-------	2,580	45	0	0	-------
735	178	do	.62	24	20	35	5.6	3,820	4,660	5,450	-------	5,360	98	0	0	-------
750	183	do	.62	24	20	-------	5.9	3,810	5,340	5,740	-------	1,790	31	0	0	-------
755	183	do	.62	24	20	35	5.7	3,500	5,190	5,650	-------	5,430	96	0	0	-------
464	109	Portland B2	.64	24	20	35	5.8	3,740	5,010	7,070	5,940	5,950	84	920	0	-------
465	110	do	.64	24	20	35	5.5	3,660	4,920	6,910	6,070	4,850	70	0	0	-------
466	111	Portland B3	.64	24	20	35	5.7	3,810	5,020	5,860	5,790	3,960	68	0	0	-------
257	63	Portland A	.59	24	20	35	5.8	2,720	4,290	5,490	6,440	3,190	58	0	0	-------
657	158	do	.62	24	20	35	6.5	4,080	4,560	4,920	-------	0	-------	0	0	-------
667	158	do	.62	24	20	-------	6.0	3,450	4,570	6,280	-------	790	13	0	0	-------
672	163	do	.62	24	20	-------	6.0	3,790	5,240	6,070	-------	1,880	31	0	0	-------
677	163	do	.62	24	20	35	5.9	3,670	4,460	6,070	-------	1,630	27	0	0	-------
692	168	do	.62	24	20	-------	5.6	3,760	5,520	5,760	-------	680	12	0	0	-------
697	168	do	.62	24	20	35	5.9	3,970	5,440	5,820	-------	1,670	29	0	0	-------
700	171	do	.62	24	20	35	5.8	3,950	5,360	6,190	-------	1,830	30	0	0	-------
710	171	do	.62	24	20	-------	5.6	4,090	4,700	5,260	-------	1,940	37	0	0	-------
731	179	do	.62	24	20	-------	5.7	3,500	4,770	6,430	-------	810	13	0	0	-------
736	179	do	.62	24	20	35	5.8	3,680	5,030	5,080	-------	2,620	52	0	0	-------
751	184	do	.62	24	20	-------	5.8	4,200	5,580	6,090	-------	2,310	38	0	0	-------
756	184	do	.62	24	20	35	5.7	3,830	5,140	5,850	-------	4,420	76	0	0	-------
369	86, 99	Portland I	.64	24	20	35	6.1	3,590	4,370	6,110	6,650	4,470	73	2,500	0	-------
703	174	do	.62	24	20	35	6.2	4,480	5,610	7,040	-------	6,240	89	4,150	-------	-------
713	174	do	.62	24	20	-------	5.7	4,370	4,790	6,330	-------	5,800	92	5,600	-------	-------
734	182	do	.62	24	20	-------	6.0	4,040	5,580	5,940	-------	6,230	105	6,740	-------	-------
739	182	do	.62	24	20	35	5.8	3,620	5,200	5,810	-------	6,930	119	6,180	-------	-------
754	187	do	.62	24	20	-------	6.2	3,980	5,050	6,770	-------	5,950	88	6,130	-------	-------
759	187	do	.62	24	20	35	5.7	3,840	5,380	6,210	-------	6,040	97	6,810	-------	-------
761	199	do	.62	24	20	35	6.0	4,160	4,830	7,560	-------	6,900	91	-------	-------	-------
372	97	Portland K2	.64	24	20	35	6.4	3,120	4,440	5,720	7,350	4,000	80	2,260	1,450	2
658	159	do	.62	24	20	35	6.5	3,990	5,170	6,290	-------	5,490	87	1,740	-------	-------
668	159	do	.62	24	20	-------	6.0	3,140	5,240	5,980	-------	5,790	97	4,410	-------	-------
673	164	do	.62	24	20	-------	6.0	4,260	5,340	6,800	-------	5,830	86	2,690	-------	-------
678	164	do	.62	24	20	35	5.9	3,770	5,580	6,270	-------	5,640	90	2,710	-------	-------
693	169	do	.62	24	20	-------	6.2	3,280	5,370	5,740	-------	5,460	95	5,060	-------	-------
698	169	do	.62	24	20	35	6.2	3,400	4,730	6,350	-------	6,580	104	4,610	-------	-------
373	98	Portland K1	.64	24	20	35	6.1	4,220	4,970	7,050	6,520	3,400	48	0	0	-------
364	81	Portland E	.62	24	20	35	5.8	3,340	4,770	7,320	7,540	2,910	40	0	0	-------
404	103	Portland S	.62	24	20	35	6.0	3,010	4,380	5,470	6,310	3,160	58	1,230	0	-------
413	105	Portland Q2	.62	24	20	35	5.7	3,520	5,390	6,500	6,350	4,300	66	0	0	-------
401	100	Portland Q1	.62	24	20	35	6.4	2,900	4,310	5,110	5,960	0	-------	0	0	-------
370	87, 88	Portland P	.64	24	20	35	6.1	3,730	5,040	6,540	6,700	810	12	0	0	-------
371	84	Portland L	.64	24	20	35	5.8	3,690	5,000	5,770	6,230	3,170	55	250	0	-------
260	34	Portland C	.66	24	20	35	6.1	3,340	4,540	5,980	6,850	5,090	85	4,390	4,290	63
367	82, 95	do	.67	24	20	35	6.8	3,220	4,320	6,270	7,760	4,790	76	2,280	0	-------
654	155	do	.66	24	20	35	6.6	3,580	5,150	6,220	-------	5,910	95	3,260	-------	-------
664	155	do	.66	24	20	-------	6.5	3,220	4,890	6,590	-------	6,250	95	4,820	-------	-------
669	160	do	.66	24	20	-------	6.8	3,150	5,180	5,900	-------	6,190	105	5,120	-------	-------
674	160	do	.66	24	20	35	6.5	3,430	5,480	7,030	-------	5,610	80	5,820	-------	-------
689	165	do	.66	24	20	-------	6.3	3,200	5,600	7,390	-------	5,670	77	4,960	-------	-------
694	165	do	.66	24	20	35	6.5	3,480	5,520	6,780	-------	5,910	87	4,400	-------	-------
365	83	Portland H	.62	24	20	35	6.1	3,940	4,770	6,280	7,250	3,680	59	290	0	-------
656	157	do	.62	24	20	35	6.5	3,230	5,090	5,670	-------	4,510	80	0	0	-------
666	157	do	.62	24	20	-------	6.0	3,520	5,120	6,640	-------	5,490	82	0	0	-------
671	162	do	.62	24	20	-------	6.7	3,380	4,690	6,260	-------	5,550	89	4,140	-------	-------
676	162	do	.62	24	20	35	6.5	3,330	4,490	5,830	-------	5,870	101	3,900	-------	-------
691	167	do	.62	24	20	-------	6.9	2,360	3,760	5,040	-------	4,150	82	610	0	-------
696	167	do	.62	24	20	35	6.6	3,080	4,570	5,510	-------	5,490	100	2,290	-------	-------
259	33	Portland D	.59	24	20	35	5.4	3,340	4,780	5,900	6,230	1,590	27	0	0	-------

TABLE 2.—*Record of tests of 2 by 4 inch concrete cylinders exposed to the action of sulphate water of Medicine Lake, S. Dak., and parallel tests of cylinders stored in tap water in laboratory tanks*—Continued

PART 2.—PORTLAND CEMENTS FROM DIFFERENT MILLS—Continued

Series No.	Cement laboratory No.	Cement	Water ratio	Curing method			Absorption at 21 days	Average of compression tests								
								Tank specimens				Lake specimens				
				Time in moist closet	Time in water	Time in air		7 days	28 days	1 year	5 years	1 year	Percentage of normal strength as indicated by tank specimens at 1 year	3 years	5 years	Percentage of normal strength as indicated by tank specimens at 5 years
				Hours	*Days*	*Days*	*Per cent.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*		*Lbs. per sq. in.*	*Lbs. per sq. in.*	
368	85, 94	Portland D	0. 64	24	20	35	6. 4	3, 390	4, 130	5, 730	6, 770	2, 980	52	0	0	
701	172	do	. 62	24	20	35	6. 1	4, 040	5, 600	6, 170		5, 750	93	2, 170		
711	172	do	. 62	24	20		5. 8	3, 950	5, 380	7, 050		4, 980	71	2, 070		
732	180	Portland D	. 62	24	20		5. 9	3, 780	5, 400	6, 620		4, 750	72	0	0	
737	180	do	. 62	24	20	35	6. 2	3, 960	5, 130	5, 580		4, 760	85	1, 160		
752	185	do	. 62	24	20		5. 8	4. 390	5, 290	6, 090		3, 850	63	0	0	
757	185	do	. 62	24	20	35	5. 7	4, 080	5, 530	6, 160		4, 860	79	3, 490		
763	196	Portland BB	. 62	24	20	35	5. 9	4, 030	5, 060	6. 360		2, 830	44			
361	80	Portland F	. 62	24	20	35	6. 3	2, 870	4, 320	6. 350	5, 730	0		0	0	
769	201	Portland X	. 62	24	20	35	6. 3	3, 050	4, 960	6, 440		4, 440	69			
576	133	Portland AA	. 62	24	20	35	6. 0	3 460	4, 820	5, 850		3, 440	59	0	0	
765	197	do	. 60	24	20	35	5. 8	3, 750	5, 430	4, 820		0		0	0	
402	101	Portland R	. 62	24	20	35	6. 3	3, 680	4, 940	6, 290	5, 900	2, 440	39	0	0	
415	107	Portland V	. 62	24	20	35	6. 5	2, 830	4, 540	5, 920	5, 550	2, 940	50	0	0	
555	128	Portland Y	. 64	24	20	35	6. 0	3, 780	4, 670	5, 480		3, 510	64	0	0	
557	128	do	. 73	24	20	35	6. 4	2, 760	4, 250	4, 690		1, 800	38	0	0	
313	65	Portland M	. 60	24	20	35	5. 9	3, 150	3. 950	6, 050	5, 680	2, 250	37	0	0	
366	77, 96	Portland G	. 64	24	20	35	6. 0	4, 050	4, 820	5, 900	6, 690	1, 930	33	0	0	
577	134	do	. 62	24	20	35	6. 3	3, 910	5, 610	6, 730		6, 140	91	3, 340		
702	173	do	. 62	24	20	35	6. 1	3, 530	5 330	6, 510		5, 290	81	3. 270		
712	173	do	. 62	24	20		5. 7	3, 630	4, 910	6, 120		5, 880	96	5, 660		
733	181	do	. 62	24	20		6. 0	4, 070	5, 190	5, 710		6, 040	106	4, 150		
738	181	do	. 62	24	20	35	5. 9	3, 810	5, 080	6, 290		5, 960	95	5, 170		
753	186	do	. 62	24	20		5. 9	3, 750	5, 420	6, 290		5, 890	94	5, 670		
758	186	do	. 62	24	20	35	5. 6	3, 860	5, 020	5, 930		5, 800	98	7, 290		
363	89	Portland O1	. 62	24	20	35	5. 9	4, 010	5, 220	6, 730	6, 430	670	10	0	0	
468	113	Portland O2	. 64	24	20	35	6. 3	2, 140	3, 940	6, 000	5, 260	5, 590	93	5, 580	3, 550	67
767	198	Portland CC	. 62	24	20	35	6. 2	3, 440	5, 540	6, 650		5, 330	80			
405	104	Portland T	. 62	24	20	35	6. 4	2, 940	4, 380	5, 530	5, 860	280	5	0	0	
416	108	Portland W	. 62	24	20	35	5. 8	3, 670	5, 010	6, 510	5, 240	1, 490	23	0	0	
414	106	Portland U	. 62	24	20	35	6. 1	3, 030	4, 410	6, 430	5, 900	2, 490	39	0	0	
362	79	Portland N1	. 62	24	20	35	6. 1	3, 220	4, 600	6, 970	6, 550	4, 000	57	950	0	
403	102	Portland N2	. 62	24	20	35	5. 9	3, 860	5, 640	6, 630	5, 720	4, 410	67	1, 860	0	
575	132	Portland Z	. 62	24	20	35	6. 0	4, 760	5, 160	6, 490		4, 970	77	2, 210		

PART 3.—SPECIAL CEMENTS OTHER THAN HIGH ALUMINA

573	130	Special B	0. 62	24	20	35	6. 3	4, 140	4, 800	5, 610		5, 880	105	3, 710		
714	175	Special D	. 64	24	20	35	5. 6	5, 080	6, 040	6, 250		5, 620	90	0	0	
574	131	Special C	. 62	24	20	35	6. 6	3, 490	4, 580	6, 100		5, 680	93	3, 360		
764	191	Special E	. 62	24	20	35	3. 8	4, 480	5, 280	5, 990		3, 570	60			
765	197	Portland AA	. 60	24	20	35	5. 8	3, 750	5, 430	4. 820		0		0	0	
762	200	Special F	. 62	24	20	35	5. 6	5, 390	6, 610	6, 660		3, 830	58			
763	196	Portland BB	. 62	24	20	35	5. 9	4, 030	5, 660	6, 360		2, 830	44			
768	193	Special G	. 62	24	20	35	5. 9	2, 910	4, 410	5, 930		4, 410	74			
769	201	Portland X	. 62	24	20	35	6. 3	3, 050	4, 960	6, 440		4, 440	69			
[1] 467	112	Special X	. 54	24	20	35		2, 830	4, 390	4, 820	4, 350	4, 830	100	3, 570	2, 640	61
[2] 719	177	do	. 44	24	20	35	10. 7	3, 940	5, 200	5, 660		4, 910	87	4, 520		
720–721	177	do	. 54	24	20	35	8. 0	3, 000	4, 400	5, 100		4, 340	85	3, 730		
[3] 722	177	do	. 61	24	20	35	6. 6	2, 690	4, 020	4, 220		3, 710	88	3, 500		
[4] 723	177	do	. 90	24	20	35	8. 2	860	1, 530	2, 230		1, 700	76	210	0	
[2 5] 724	177	do	. 42	24	20	35	9. 5	4, 280	5, 900	6, 680		6, 070	91	6, 220		
[3 5] 725–726	177	do	. 51	24	20	35	9. 4	2, 450	4, 080	4, 240		3, 960	93	3, 760		
[3 5] 727	177	do	. 70	24	20	35	13. 3	1, 200	2, 370	3, 050		2, 150	70	1, 400		
[4 5] 728	177	do	1. 08	24	20	35	15. 8	320	780	990		0		0	0	
766	192	Special H	. 62	24	20	35	6. 0	6, 360	6, 480	7, 570		6, 520	86			
767	198	Portland CC	. 62	24	20	35	6. 2	3, 440	5, 540	5, 650		5, 330	80			
760	190	Special I	. 64	24	20	35	4. 7	4, 920	5, 570	5, 780		4, 810	83			
761	199	Portland I	. 62	24	20	35	6. 0	4, 160	4, 830	7, 560		6, 900	91			
243–247	28	Special A1	. 62	24	20	35	6. 2	2, 910	3, 770	5, 370	5, 960	4, 030	75	0	0	
248–250	10	Special A3	. 63	24	20	35	9. 5	1, 430	2, 730	3, 750	3, 620	3, 280	87	800	0	
556	127	Special A2	. 64	24	20	35	6. 0	3, 200	4, 720	5, 950		5, 330	90	0	0	
555	128	Portland Y	. 64	24	20	35	6. 0	3 780	4, 670	5, 480		3, 510	64	0	0	
558	127	Special A2	. 73	24	20	35	6. 6	2, 280	4, 480	5, 290		3, 080	58	0	0	
557	128	Portland Y	. 73	24	20	35	6. 4	2, 760	4, 250	4, 690		1, 800	38	0	0	
[6] 561	127	Special A2	. 62	24	[7] 20	35	7. 9	3, 260	4, 720	5, 860		6, 090	104	4, 120		
[6] 560	128	Portland Y	. 62	24	[7] 20	35	8. 2	3, 550	5, 160	6, 120		5, 270	86	4, 040		
[6] 563	127	Special A2	. 71	24	[7] 20	35	9. 1	2, 290	4, 520	4, 960		3, 960	80	1, 140		
[6] 562	128	Portland Y	. 71	24	[7] 20	35	9. 2	2, 720	4, 290	5, 450		3, 920	72	900		

[1] Mix — 1: 1.88.
[2] Mix — 1: 0.94.
[3] Mix — 1: 2.82.
[4] Mix — 1: 4.70.
[5] Standard Ottawa sand cylinders.
[6] Mix—1: 2.25.
[7] In damp sand.

TABLE 2.—*Record of tests of 2 by 4 inch concrete cylinders exposed to the action of sulphate water of Medicine Lake, S. Dak., and parallel tests of cylinders stored in tap water in laboratory tanks*—Continued

PART 4.—HIGH ALUMINA CEMENTS

Series No.	Cement laboratory No.	Cement	Water ratio	Curing method					Average of compression tests								
										Tank specimens				Lake specimens			
				Time in moist closet	Time in water	Time in water vapor or steam	Temperature of water vapor or steam	Time in air	Absorption at 21 days	7 days	28 days	1 year	5 years	1 year	Percentage of normal strength as indicated by tank specimens at 1 year	2 or 3 years	5 years	Percentage of normal strength as indicated by tank specimens at 5 years
				Hours	*Days*	*Hours*	*°F.*	*Days*	*Per cent*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*		*Lbs. per sq. in.*	*Lbs. per sq. in.*	
137	14	High alumina A	0.64	24	------	48	155	25	6.2	3,250	3,550	2,830	2,850	3,400	120	4,510	4,290	151
138	14	------do	.64	24	------	24 / 24	155 / 212	25	6.2	3,150	3,670	2,860	2,510	3,320	116	------	4,960	198
139	14	------do	.64	24	------	72	155	24	6.1	3,160	3,840	2,640	2,460	3,790	144	------	4,560	185
140	14	------do	.64	24	------	48 / 24	155 / 212	24	6.1	3,190	3,890	2,730	2,570	3,790	139	------	5,040	196
141	14	------do	.64	24	------	24 / 48	155 / 212	24	6.1	3,360	3,800	3,020	2,700	3,670	121	------	4,920	182
147	14	------do	.62	24	27	------	------	------	6.3	4,560	4,890	4,560	3,460	5,220	114	[1] 6,410	4,460	129
148	14	------do	.62	72	------	------	------	25	7.0	5,020	4,810	5,310	3,710	5,960	112	[1] 6,300	6,230	168
149	14	------do	.62	24	------	48	100	25	6.5	5,370	5,510	5,240	2,650	5,450	104	[1] 5,300	5,330	201
150	14	------do	.62	24	------	48	155	25	6.2	3,360	3,870	------	2,640	3,400	------	[1] 4,060	3,690	140
151	14	------do	.62	24	------	48	212	25	6.1	3,300	3,750	2,840	2,660	3,650	129	[1] 4,110	4,090	154
152	14	------do	.62	24	20	------	------	344	6.3	4,570	5,040	5,100	3,780	5,040	99	------	6,240	165
153	14	------do	.62	24	20	------	------	56	6.5	4,830	4,910	5,120	3,690	6,280	123	------	6,720	182
154	14	------do	.62	24	20	------	------	28	6.5	4,760	4,850	5,450	3,380	5,620	103	------	6,460	191
155	14	------do	.62	24	20	------	------	14	6.5	4,250	4,960	4,570	3,590	5,770	126	------	6,420	179
156	14	------do	.62	24	20	------	------	------	6.3	4,710	4,860	4,380	3,160	4,960	113	------	4,390	139
132	14	------do	.65	24	------	48	155	25	8.1	2,250	2,910	2,760	2,830	2,790	101	[1] 2,930	2,940	104
133	14	------do	.65	24	------	24 / 24	155 / 212	25	8.0	2,280	2,680	2,080	2,690	2,900	139	[1] 3,140	2,840	106
134	14	------do	.65	24	------	72	155	24	8.0	2,470	2,850	2,440	2,640	2,840	116	[1] 3,120	3,090	117
135	14	------do	.65	24	------	48 / 24	155 / 212	24	7.8	2,620	2,960	2,650	2,910	2,590	98	[1] 2,930	3,340	115
136	14	------do	.65	24	------	24 / 48	155 / 212	24	7.8	2,550	2,930	2,630	2,650	2,580	98	[1] 3,170	3,030	114
142	14	------do	.63	24	27	------	------	------	8.7	2,910	3,600	3,590	2,240	4,060	113	------	3,640	162
143	14	------do	.63	72	------	------	------	25	9.3	4,220	4,430	4,220	2,160	4,310	102	------	5,880	272
144	14	------do	.63	24	------	48	100	25	9.1	4,320	4,640	3,750	1,880	4,490	120	------	4,120	219
145	14	------do	.63	24	------	48	155	25	8.1	2,200	2,760	2,780	2,930	2,620	94	------	3,760	128
146	14	------do	.63	24	------	48	212	25	7.8	2,500	2,870	2,500	2,400	3,010	120	------	3,960	165
208–212	25	High Alumina B	.51	24	20	------	------	35	4.9	6,700	6,780	6,260	7,770	7,420	118	7,240	8,950	115
[2] [3] 276	27	------do	.44	24	20	------	------	35	6.3	6,210	7,680	6,490	6,390	8,000	123	7,370	7,970	125
277–278	27	------do	.59	24	20	------	------	35	8.3	5,060	4,830	5,110	3,640	5,560	109	5,350	5,690	156
[2] [4] 279	27	------do	.73	24	20	------	------	35	10.2	3,380	3,090	3,250	2,380	3,730	115	3,350	3,830	161
[2] [5] 280	27	------do	.99	24	20	------	------	35	11.8	2,160	1,920	1,680	1,130	700	42	0	0	------
[3] 286	27	------do	.44	24	20	------	------	35	5.1	7,490	6,820	8,080	8,100	8,460	105	8,080	8,210	101
287–288	27	------do	.53	24	20	------	------	35	4.9	7,080	6,620	8,160	6,880	8,210	101	6,710	7,830	114
[4] 289	27	------do	.67	24	20	------	------	35	5.3	6,560	5,680	6,290	4,090	6,160	98	5,460	6,310	154
[5] 290	27	------do	.81	24	20	------	------	35	5.8	4,640	4,840	4,150	2,520	5,080	122	4,080	4,770	189
396	70, 71	------do	.44	24	20	------	------	35	5.6	5,300	5,760	6,110	4,040	5,780	95	6,040	5,920	147
397	70, 71	------do	.53	24	20	------	------	35	4.9	6,660	7,120	8,100	7,040	7,580	94	8,660	7,710	110
398	70, 71	------do	.59	24	20	------	------	35	5.3	6,890	7,210	8,630	6,280	7,390	86	8,300	6,780	108
399	70, 71	------do	.73	24	20	------	------	35	6.3	5,950	5,710	6,440	3,700	5,400	84	6,690	5,790	156
400	70, 71	------do	.88	24	20	------	------	35	7.6	5,140	4,230	3,480	2,550	3,790	109	2,950	3,200	125
[6] 429	70, 71	------do	.35	24	20	------	------	35	9.7	9,460	8,420	10,440	6,620	9,330	89	10,160	8,510	129
435	70, 71, 74	High alumina B, 5 per cent, Portland B, 47.5 per cent, and Portland A, 47.5 per cent	.66	24	20	------	------	35	6.3	2,630	3,880	5,490	5,970	4,100	75	0	0	------
436	70, 71, 74	High alumina B, 10 per cent, Portland B, 45 per cent, and Portland A, 45 per cent	.66	24	20	------	------	35	6.2	2,260	3,760	4,820	5,420	2,720	56	0	0	------
437	70, 71, 74	High alumina B 20 per cent, Portland B, 40 per cent, and Portland A, 40 per cent	.66	24	20	------	------	35	7.0	1,360	2,830	3,640	3,060	3,320	91	0	0	------
438	70, 71	High alumina B	.66	24	20	------	------	35	5.8	7,080	7,790	7,820	6,440	7,220	92	8,130	7,810	121
252–253	58	High alumina C	.53	24	20	------	------	35	5.4	7,730	7,230	7,480	4,030	7,010	94	7,230	7,340	182
[5] 291	58	------do	.79	24	20	------	------	35	6.5	4,360	4,630	2,910	2,190	4,370	150	3,330	3,410	156
[2] 254–255	58	------do	.60	24	20	------	------	35	9.3	5,010	4,850	2,920	2,020	4,340	149	3,690	3,170	157
[2] [5] 292	58	------do	.90	24	20	------	------	35	12.3	2,130	1,820	1,130	1,230	920	81	0	0	------

[1] Two-year tests. [2] Standard Ottawa sand cylinders. [3] Mix — 1:2. [4] Mix — 1:4. [5] Mix — 1:5. [6] Neat cement.

TABLE 2.—*Record of tests of 2 by 4 inch concrete cylinders exposed to the action of sulphate water of Medicine Lake, S. Dak., and parallel tests of cylinders stored in tap water in laboratory tanks*—Continued

PART 5.—SURFACE TREATMENT OR IMPREGNATION

Series No.	Cement laboratory No.	Cement	Water ratio	Surface treatment or impregnation	Curing method			Absorption at 21 days	Average of compression tests								
									Tank specimens				Lake specimens				
					Time in moist closet	Time in water	Time in air		7 days	28 days	1 year	5 years	1 year	Percentage of normal strength as indicated by tank specimens at 1 year	3 years	5 years	Percentage of normal strength as indicated by tank specimens at 5 years
					Hours	*Days*	*Days*	*Per cent*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*		*Lbs. per sq. in.*	*Lbs. per sq. in.*	
374	74	½ Portland A, ½ Portland B.	0.62	Inertol, first coat at 22 days, second at 26 days.	24	20	35	6.7	2,930	4,870	5,570	6,200	5,390	97	3,310	5,320	86
[1] 375	74	do	.64	do	24	20	35	10.5	1,430	2,790	3,640	3,740	2,970	82	0	0	
391	74	do	.62	No treatment	24	20	35	6.3	3,130	4,890	6,060	6,430	2,480	41	0	0	
549	129	do	.62	No treatment. Dipped in boiling water ½ minute at 28 days and ½ minute at 31 days.	24	20	35	5.8	3,790	4,770	6,550		1,620	25	0	0	
550	129	do	.62	Boiled linseed oil at 70° F., dipped ½ minute at 28 days.	24	20	35	5.8	3,540	4,960	6,040		5,520	91	5,220		
551	129	do	.62	Boiled linseed oil at 70° F., dipped ½ minute at 28 days and ½ minute at 31 days.	24	20	35	5.8	3,730	5,260	5,520		5,730	104	5,560		
552	129	do	.62	Boiled linseed oil at 225° F., dipped ½ minute at 28 days.	24	20	35	5.8	3,430	4,680	5,970		5,330	89	5,260		
553	129	do	.62	Boiled linseed oil at 225° F., dipped ½ minute at 28 days and ½ minute at 31 days.	24	20	35	5.8	3,750	4,960	5,510		5,650	103	5,050		
554	129	do	.64	No treatment	24	20	35	6.3	3,570	4,760	5,980		3,530	59	0	0	
934	245	do	.62	McEverlast "Special Paving Coating," one brush coat at 24 hours.	24		55	4.0	4,370	5,270	6,030		5,610	93			
935	245	do	.62	McEverlast "Special Paving Coating," one brush coat at 21 days.	24	20	35	5.8	4,340	5,640	5,440		5,710	105			
976	237	do	.62	No treatment	24	20	35	6.0	3,700	4,590	5,750		4,650	81			
977	237	do	.62	McEverlast "Penetration," one brush coat at 24 hours, followed by one brush coat, "Concrete Cover Coat" at 48 hours.	24		55	2.7	2,890	4,180	5,660		4,710	83			
978	237	do	.62	McEverlast "Penetration," one brush coat at 21 days followed by one brush coat, "Concrete Cover Coat" at 22 days.	24	20	35	2.1	3,670	4,650	5,740		5,550	97			
979	237	do	.62	McEverlast "Paving Special," one brush coat at 24 hours followed by one brush coat "Concrete Cover Coat" at 48 hours.	24		55	4.5	3,200	4,410	5,620		4,920	88			
980	237	do	.62	McEverlast "Paving Special," one brush coat at 21 days followed by one brush coat "Concrete Cover Coat" at 22 days.	24	20	35	2.0	3,410	4,450	5,780		5,250	91			
[1] 293	19	do	.64	No treatment	24	20	35	9.9	1,420	2,240	3,850	3,740	2,140	56	0	0	
[1] 294	19	do	.64	Sulphur impregnated	24	20	35	9.9	1,350	2,210	3,130	2,610	1,500	48	0	0	

[1] Standard Ottawa sand cylinders.

TABLE 2.—*Record of tests of 2 by 4 inch concrete cylinders exposed to the action of sulphate water of Medicine Lake, S. Dak., and parallel tests of cylinders stored in tap water in laboratory tanks*—Continued

PART 6.—ADMIXTURES

Series No.	Cement laboratory No.	Cement	Water ratio	Admixture	Curing method						Average of compression tests							
											Tank specimens				Lake specimens			
					Time in moist closet	Time in water or damp sand	Time in water vapor or steam	Temperature of water vapor or steam	Time in air	Absorption at 21 days	7 days	28 days	1 year	5 years	1 year	Percentage of normal strength as indicated by tank specimens at 1 year	2 or 3 years	5 years	Percentage of normal strength as indicated by tank specimens at 5 years
					Hours	*Days*	*Hours*	*°F.*	*Days*	*Per cent*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*		*Lbs. per sq. in.*	*Lbs. per sq. in.*	
182	18	½ Portland A, ½ Portland B.	0.59	None	24	20	-----	-----	35	6.0	2,800	4,010	5,780	5,790	4,450	77	[1]2,100	0	-----
183	18	----do	.63	3 per cent Alkagel A.	24	20	-----	-----	35	5.3	2,260	3,560	4,570	4,670	3,520	77	[1]2,140	0	-----
184	18	----do	.59	None	24	-----	48	155	35	6.0	4,070	4,530	5,100	6,110	4,240	83	[1]4,700	800	13
185	18	----do	.63	3 per cent Alkagel A.	24	-----	48	155	35	5.0	3,280	3,940	4,570	5,050	3,300	72	[1]3,900	1,090	22
186	18	----do	.63	----do	24	-----	48	100	35	5.4	3,120	3,730	3,820	4,420	3,550	93	[1]3,060	0	-----
554	129	----do	.64	None	24	20	-----	-----	35	6.3	3,570	4,760	5,980	-----	3,530	59	0	0	-----
581	129	----do	.62	6 per cent barium chloride.	24	20	-----	-----	35	6.5	3,530	4,570	5,870	-----	3,730	64	0	0	-----
582	129	----do	.62	12 per cent barium chloride.	24	20	-----	-----	35	6.7	3,240	4,460	5,100	-----	4,420	87	2,800	0	-----
822	225	----do	.62	None	24	20	-----	-----	35	5.3	3,220	5,230	6,250	-----	3,800	61	-----	-----	-----
823	225	----do	.62	3.75 per cent Barnsdall.	24	20	-----	-----	35	5.2	3,440	4,660	7,090	-----	3,050	43	-----	-----	-----
824	225	----do	.64	7.5 per cent Barnsdall.	24	20	-----	-----	35	5.5	2,970	5,070	6,480	-----	3,000	46	-----	-----	-----
825	225	----do	.72	15.0 per cent Barnsdall.	24	20	-----	-----	35	6.0	3,060	4,490	5,350	-----	2,130	40	-----	-----	-----
826	225	----do	.79	30.0 per cent Barnsdall.	24	20	-----	-----	35	6.8	2,310	3,790	5,410	-----	1,450	27	-----	-----	-----
[2]981	263	----do	.64	None	24	27	-----	-----	28	5.9	-----	5,440	7,100	-----	5,030	71	-----	-----	-----
[2]982	263	----do	.67	4 per cent Barnsdall.	24	27	-----	-----	28	6.1	-----	5,630	6,610	-----	4,590	69	-----	-----	-----
[2]983	263	----do	.69	6 per cent Barnsdall.	24	27	-----	-----	28	6.0	-----	5,140	6,050	-----	4,060	67	-----	-----	-----
[2]984	263	----do	.71	8 per cent Barnsdall.	24	27	-----	-----	28	6.1	-----	5,560	5,950	-----	3,800	64	-----	-----	-----
[2]985	274	Portland I	.64	None	24	27	-----	-----	28	5.9	-----	5,330	7,190	-----	7,170	100	-----	-----	-----
[2]986	274	----do	.67	4 per cent Barnsdall.	24	27	-----	-----	28	6.0	-----	5,370	7,890	-----	6,170	78	-----	-----	-----
[2]987	274	----do	.69	6 per cent Barnsdall.	24	27	-----	-----	28	6.1	-----	5,480	7,680	-----	5,460	71	-----	-----	-----
[2]988	274	----do	.71	8 per cent Barnsdall.	24	27	-----	-----	28	6.3	-----	5,310	7,080	-----	6,610	93	-----	-----	-----
[3]989	263	½ Portland A, ½ Portland B.	.80	None	24	27	-----	-----	28	6.2	-----	4,850	5,540	-----	1,570	28	-----	-----	-----
[3]990	263	----do	.82	4 per cent Barnsdall.	24	27	-----	-----	28	6.6	-----	5,010	5,770	-----	630	11	-----	-----	-----
[3]991	263	----do	.83	6 per cent Barnsdall.	24	27	-----	-----	28	6.4	-----	4,770	5,340	-----	980	18	-----	-----	-----
[3]992	263	----do	.84	8 per cent Barnsdall.	24	27	-----	-----	28	6.2	-----	4,660	6,050	-----	570	9	-----	-----	-----
[3]993	274	Portland I	.80	None	24	27	-----	-----	28	6.5	-----	3,980	6,610	-----	3,970	60	-----	-----	-----
[3]994	274	----do	.82	4 per cent Barnsdall.	24	27	-----	-----	28	6.4	-----	4,740	6,540	-----	4,650	71	-----	-----	-----
[3]995	274	----do	.83	6 per cent Barnsdall.	24	27	-----	-----	28	6.8	-----	4,440	5,990	-----	4,850	81	-----	-----	-----
[3]996	274	----do	.84	8 per cent Barnsdall.	24	27	-----	-----	28	6.6	-----	4,500	6,010	-----	4,260	71	-----	-----	-----
[4]997	263	½ Portland A, ½ Portland B.	.94	None	24	27	-----	-----	28	6.7	-----	3,640	4,370	-----	2,840	65	-----	-----	-----
[4]998	263	----do	.95	4 per cent Barnsdall.	24	27	-----	-----	28	6.8	-----	3,570	4,640	-----	2,700	58	-----	-----	-----
[4]999	263	----do	.96	6 per cent Barnsdall.	24	27	-----	-----	28	6.9	-----	4,020	4,510	-----	1,510	33	-----	-----	-----
[4]1000	263	----do	.97	8 per cent Barnsdall.	24	27	-----	-----	28	6.9	-----	3,610	4,650	-----	830	18	-----	-----	-----
[4]1001	274	Portland I	.94	None	24	27	-----	-----	28	6.5	-----	3,370	4,880	-----	4,360	89	-----	-----	-----
[4]1002	274	----do	.95	4 per cent Barnsdall.	24	27	-----	-----	28	6.6	-----	3,480	4,730	-----	4,080	86	-----	-----	-----
[4]1003	274	----do	.96	6 per cent Barnsdall.	24	27	-----	-----	28	6.6	-----	3,560	4,360	-----	3,950	91	-----	-----	-----
[4]1004	274	----do	.97	8 per cent Barnsdall.	24	27	-----	-----	28	6.7	-----	3,570	4,780	-----	5,020	105	-----	-----	-----
157	18	½ Portland A, ½ Portland B.	0.74	40 per cent blast furnace slag.	24	20	-----	-----	-----	6.5	2,420	3,960	6,030	6,200	4,230	70	[1]2,340	0	-----
158	18	----do	.63	10 per cent blast furnace slag.	24	20	-----	-----	35	6.0	2,880	4,180	5,440	6,260	5,170	95	[1]3,930	0	-----
159	18	----do	.74	40 per cent blast furnace slag.	24	20	-----	-----	35	6.7	2,410	4,190	5,670	5,600	5,560	98	[1]5,220	690	12
160	18	----do	.63	10 per cent blast furnace slag.	24	-----	48	155	35	6.4	3,850	4,010	5,300	4,060	4,800	91	[1]3,540	0	-----
161	18	----do	.74	40 per cent blast furnace slag.	24	-----	48	155	35	6.9	3,800	4,020	5,450	5,710	4,830	89	[1]5,640	2,760	48
172	18	----do	.59	None	24	20	-----	-----	35	5.6	2,870	4,190	5,300	5,400	4,370	82	[1]2,050	0	-----

[1] Two-year tests.
[2] Special high silica aggregate, 1:3 Mix. Two by four inch cylinders cured in damp sand.
[3] Special high silica aggregate, 1:2:3 Mix. Two by four inch cylinders cured in damp sand.
[4] Special high silica aggregate, 1:2:4 Mix. Four by eight inch cylinders cured in damp sand.

TABLE 2.—*Record of tests of 2 by 4 inch concrete cylinders exposed to the action of sulphate water of Medicine Lake, S. Dak., and parallel tests of cylinders stored in tap water in laboratory tanks*—Continued

PART 6.—ADMIXTURES—Continued

Series No.	Cement laboratory No.	Cement	Water ratio	Admixture	Curing method					Absorption at 21 days	Average of compression tests								
											Tank specimens				Lake specimens				
					Time in moist closet	Time in water or damp sand	Time in water vapor	Temperature of water vapor	Time in air		7 days	28 days	1 year	5 years	1 year	Percentage of normal strength as indicated by tank specimens at 1 year	2 or 3 years	5 years	Percentage of normal strength as indicated by tank specimens at 5 years
					Hours	*Days*	*Hours*	*°F.*	*Days*	*Per cent*	*Lbs. per sq. in.*	*Lbs. per sq. in*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*		*Lbs. per sq. in.*	*Lbs. per sq. in.*	
167	18	½ Portland A, ½ Portland B.	0.59	4 per cent Cal	24	20				5.7	3,200	4,150	4,860	5,660	3,910	80	[1]1,380	0	25
168	18	do	.60	8 per cent Cal	24	20			35	5.8	3,180	4,290	5,570	6,370	4,620	83	[1]3,690	1,570	
169	18	do	.59	4 per cent Cal	24	20			35	5.8	3,250	4,280	4,550	5,580	4,800	105	[1]3,620	1,090	20
170	18	do	.60	8 per cent Cal	24		48	155	35	6.0	4,990	4,670	5,030	5,960	5,010	100	[1]6,200	5,540	93
171	18	do	.59	4 per cent Cal	24		48	155	35	6.0	4,550	4,690	5,140	5,730	4,980	97	[1]5,560	4,690	82
172	18	do	.59	None	24	20			35	5.6	2,870	4,190	5,300	5,400	4,370	82	[1]2,050	0	
162	18	do	.63	4 per cent calcium chloride.	24	20				5.7	2,870	4,330	5,530	5,130	3,910	71	[1]1,710	0	
163	18	do	.79	8 per cent calcium chloride.	24	20			35	7.1	2,200	3,010	4,280	4,220	3,550	83	[1]3,130	1,040	25
164	18	do	.63	4 per cent calcium chloride.	24	20			35	5.9	2,940	4,000	5,450	5,790	5,050	93	[1]5,240	2,200	38
165	18	do	.79	8 per cent calcium chloride.	24		48	155	35	7.1	3,800	3,640	4,600	4,620	3,300	72	[1]3,860	3,770	82
166	18	do	.63	4 per cent calcium chloride.	24		48	155	35	5.8	4,840	4,920	4,990	5,970	4,460	89	[1]5,840	4,990	84
172	18	do	.59	None	24	20			35	5.6	2,870	4,190	5,300	5,400	4,370	82	[1]2,050	0	
412	105	Portland Q2	.66	2½ per cent Celite	24	20			35	6.4	3,530	5,050	6,670	6,280	3,760	56	0	0	
413	105	do	.62	None	24	20			35	5.7	3,520	5,390	6,500	6,350	4,300	66	0	0	
583	139	½ Portland A, ½ Portland B.	.62	do	24	20			35	6.3	3,810	4,950	6,420		3,650	57	0	0	
634	139	do	.64	2 per cent Celite	24	20			35	6.1	3,260	5,340	6,180		5,290	86	0	0	
635–636	139	do	.64	None	24		48	212	53	6.3	3,960	4,330	4,250		4,370	103	5,950		
637–638	139	do	.64	2 per cent Celite	24		48	212	53	6.2	4,160	4,730	5,300		4,410	83	5,020		
639	139	do	.64	2 per cent Colloy	24	20			35	6.5	3,630	4,610	5,250		4,630	88	0	0	
640	139	do	.64	None	24	20			35	6.5	3,320	4,470	5,800		4,530	78	0	0	
641	139	do	.64	do	24		48	212	53	6.9	3,590	3,760	4,530		3,740	83	5,360		
642–643	139	do	.64	2 per cent Colloy	24		48	212	53	6.8	3,870	4,100	4,420		3,800	86	5,510		
776	204	do	.62	None	24	20			35	6.2	3,400	4,420	6,480		0		0	0	
777	204	do	.64	2 per cent Colloy	24	20			35	6.4	3,160	4,640	5,270		0		0	0	
778	204	do	.65	4 per cent Colloy	24	20			35	6.4	3,200	4,540	6,110		0		0	0	
715	176	do	.62	None	24	20			35	5.6	3,830	5,100	6,420		4,550	71	0	0	
716	176	do	.62	0.27 per cent Earthcrete.	24	20			35	5.6	4,460	5,160	5,780		4,630	80	0	0	
717	176	do	.62	1.06 per cent Earthcrete.	24	20			35	5.8	3,530	5,120	6,950		4,860	70	0	0	
718	176	do	.62	0.27 per cent Earthcrete.	24		48	155	53	5.6	5,210	5,550	5,370		5,150	96	0	0	
554	129	do	.64	None	24	20			35	6.3	3,570	4,760	5,980		3,530	59	0	0	
564–565	129	do	.62	2½ per cent fuel ash.	24	20			35	6.3	3,580	5,100	6,710		2,840	42	0	0	
566	129	do	.64	5 per cent fuel ash	24	20			35	6.4	3,300	4,760	5,850		2,670	46	0	0	
567–568	129	do	.67	10 per cent fuel ash	24	20			35	6.3	3,390	5,080	6,570		3,660	56	0	0	
172	18	do	.59	None	24	20			35	5.6	2,870	4,190	5,300	5,400	4,370	82		0	
173	18	do	.61	20 per cent ironite	24	20			35	5.9	2,950	3,980	4,890	5,800	5,020	102		1,590	27
174	18	do	.59	None	24		48	155	35	6.1	4,020	4,390	4,330	5,650	3,910	90		2,740	48
175	18	do	.61	20 per cent ironite	24		48	155	35	6.1	3,880	4,440	5,510	5,630	4,630	84		5,280	94
176	18	do	.61	do	24		48	100	35	6.1	3,680	4,220	5,250	5,770	5,190	99		5,020	87
1141	287	do	.62	None	24	20			35	5.6	4,560	5,400							
1142	287	do	.62	1 per cent kerosene	24	20			35	5.3	3,930	5,760							
1143	287	do	.62	2 per cent kerosene	24	20			35	5.1	3,790	5,740							
1144	287	do	.62	4 per cent kerosene	24	20			35	4.4	3,800	5,530							
1145	287	do	.62	8 per cent kerosene	24	20			35	4.8	3,740	4,870							
769	201	Portland X	.62	None	24	20			35	6.3	3,050	4,960	6,440		4,440	69			
770	201	do	.62	2 per cent Medusa Waterproofing.	24	20			35	5.0	3,220	4,750	5,930		3,620	51			
779	201	do	.62	1 per cent Medusa Waterproofing.	24	20			35	5.2	3,030	4,690	5,480		2,750	50			
780	201	do	.65	4 per cent Medusa Waterproofing.	24	20			35	4.0	2,780	4,700	5,340		2,190	41			

[1] 2-year tests.

TABLE 2.—*Record of tests of 2 by 4 inch concrete cylinders exposed to the action of sulphate water of Medicine Lake, S. Dak., and parallel tests of cylinders stored in tap water in laboratory tanks*—Continued

PART 6.—ADMIXTURES—Continued

Series No.	Cement laboratory No.	Cement	Water ratio	Admixture	Curing method					Average of compression tests							
										Tank specimens				Lake specimens			
					Time in moist closet	Time in water or damp sand	Time in water vapor	Temperature of water vapor	Time in air	Absorption at 21 days	7 days	28 days	1 year	5 years	1 year	Percentage of normal strength as indicated by tank specimens at 1 year	2 or 3 years	5 years	Percentage of normal strength as indicated by tank specimens at 5 years
					Hours	*Days*	*Hours*	*°F.*	*Days*	*Per cent*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*		*Lbs. per sq. in.*	*Lbs. per sq. in.*	
1146	287	½ Portland A, ½ Portland B.	0.62	None	24	20			35	5.3	4,280	5,960							
1147	287	do	.62	1 per cent oil	24	20			35	2.8	4,580	5,260							
1148	287	do	.62	2 per cent oil	24	20			35	2.4	4,460	5,990							
1149	287	do	.62	4 per cent oil	24	20			35	2.0	4,180	5,530							
1150	287	do	.62	8 per cent oil	24	20			35	1.8	3,390	4,720							
771	204	do	.62	None	24	20			35	5.9	3,450	4,440	5,870		0		0	0	
772	204	do	.65	3.75 per cent Omicron.	24	20			35	6.1	3,510	4,480	6,010		0		0	0	
773	204	do	.69	7.5 per cent Omicron.	24	20			35	6.5	3,240	4,710	5,370		0		0	0	
774	204	do	.74	15.0 per cent Omicron.	24	20			35	7.6	3,430	4,360	5,510		3,440	62			
775	204	do	.86	30.0 per cent Omicron.	24	20			35	9.9	3,050	4,220	4,980		4,430	89			
[5] 293	19	do	.64	None	24	20			35	9.9	1,420	2,240	3,850	3,740	2,140	56	0	0	
[5] 295	19	do	.67	10 per cent sulphur	24	20			35	9.7	1,150	1,250	1,810	2,610	0		0	0	
391	74	do	.62	None	24	20			35	6.3	3,130	4,890	6,060	6,430	2,480	41	0	0	
392	74	do	.76	33 per cent Trass	24	20			35	7.3	2,710	3,830	5,790	5,620	5,040	87	2,810	1,750	31
[6] 392a	74	do	.85	do	24	20			35	7.1	2,430	3,450	5,260	5,540	4,170	79	2,060	1,300	23
393	74	do	.93	66 per cent Trass	24	20			35	9.1	1,930	3,350	4,600	5,100	4,400	96	3,510	2,140	42
[7] 393a	74	do	1.09	do	24	20			35	8.4	1,440	2,670	4,270	4,740	3,660	86	2,650	1,830	39
394	86	Portland I	.78	33 per cent Trass	24	20			35	6.8	3,330	4,690	6,620	7,180	5,300	80	2,940	1,260	18
[6] 394a	86	do	.87	do	24	20			35	6.7	3,000	3,850	6,170	5,410	4,140	67	1,080	0	
395	86	do	.95	66 per cent Trass	24	20			35	7.6	2,710	3,770	6,060	4,970	4,300	71	2,570	2,110	42
[7] 395a	86	do	1.09	do	24	20			35	7.3	2,280	3,380	4,090	4,990	3,810	93	1,140	1,620	32
554	129	½ Portland A, ½ Portland B.	.64	None	24	20			35	6.3	3,570	4,760	5,980		3,530	59	0	0	
578	129	do	.62	1 per cent Truscon	24	20			35	4.7	3,370	4,700	5,320		3,750	70	0	0	
579	129	do	.62	2 per cent Truscon	24	20			35	4.1	3,370	4,580	4,980		3,280	66	0	0	
580	129	do	.62	4 per cent Truscon	24	20			35	3.7	3,020	3,980	4,630		2,420	52	0	0	
177	18	do	.59	None	24	20			35	5.9	2,520	4,200	4,930	6,120	3,670	74	[1] 1,910	0	
178	18	do	.74	20 per cent volcanic ash.	24	20			35	6.8	1,900	3,210	5,450	5,670	2,960	54	[1] 1,970	0	
179	18	do	.59	None	24		48	155	35	5.8	4,110	4,510	5,210	6,460	3,820	73	[1] 2,380	1,750	27
180	18	do	.74	20 per cent volcanic ash.	24		48	155	35	7.6	3,240	3,340	4,440	5,560	3,600	81	[1] 4,510	1,760	32
181	18	do	.74	do	24		48	100	35	7.3	2,710	2,820	5,130	5,670	4,290	84	[1] 3,590	2,430	34

[1] 2-year tests. [5] Standard Ottawa sand cylinders. [6] Mix 1 : 4. [7] Mix 1 : 5.

GASOLINE TAXES, 1930

Total tax earned on motor vehicle fuel, etc., refunds, disposition of fund, and gallons taxed

[From reports of State authorities]

State	Gross tax assessed prior to deduction of refund	Exemption refund: (Deducted from gross tax)	Total tax earning on fuel for motor vehicles [1]	Other receipts under tax law (licenses)	Grand total earning (tax and other receipts)	Disposition of grand total earnings					Tax rates, 1930			Net gallons of gasoline taxed, and used by motor vehicles
						Collection cost [2]	Construction and maintenance on rural roads		State and county road bond payments [3]	For miscellaneous purposes	Cents per gallon		Date of rate change	
							State highways	Local roads			Jan. 1	Dec. 31		
Alabama	$6,901,491		$6,901,491	$1	$6,901,492	$34,064	$1,958,478	$3,439,493	$1,469,457		4	4		172,537,281
Arizona	3,011,844	$341,825	2,670,019		2,670,019	[4]	1,682,596	987,423			4	4		66,750,478
Arkansas	6,792,273	275,000	6,427,273		6,427,273	47,908	4,505,108		[5] 1,802,755	[6] $71,502	5	5		128,545,469
California	38,603,808	3,733,682	34,870,126		34,870,126	[7] 51,532	23,212,396	11,606,198			3	3		1,162,337,545
Colorado	6,834,198	689,372	6,144,826		6,144,826	[8] 54,750	4,263,053	1,644,321		[9] 182,702	4	4		153,620,645
Connecticut		[10]	4,465,933	49,130	4,515,063	[11]	4,515,063				2	2		223,296,627
Delaware	1,071,363	58,006	1,013,357		1,013,357		864,170		149,187		3	3		33,778,561
Florida	13,622,215		13,622,215	32,960	13,655,175	13,280	4,540,738	756,790	[12] 4,540,738	[13] 3,803,629	6	6		227,036,915
Georgia	13,391,079		13,391,079	43,983	13,435,062	4,200	8,953,908	2,238,477		[14] 2,238,477	6	6		223,184,648
Idaho	2,941,124	272,542	2,668,582	62,280	2,730,862	12,752	2,674,080		35,609	[15] 8,421	4	5	Mar. 1	[16] 54,422,752
Illinois [17]	28,612,269	1,139,849	27,472,420		27,472,420	61,724	18,273,797	9,136,899			3	3		915,747,319
Indiana	18,247,078	1,088,332	17,158,746	88	17,158,834	40,958	12,838,407	3,209,602		[15] 1,069,867	4	4		428,968,653
Iowa	11,793,452	1,209,384	10,584,068		10,584,068	32,524	[19] 5,242,393	5,309,151			3	3		352,802,277
Kansas	10,853,021	1,732,530	9,120,491		9,120,491	[20]	7,370,491	1,750,000			3	3		304,016,374
Kentucky	8,414,733		8,414,733		8,414,733	28,364	8,386,369				5	5		168,294,655
Louisiana	7,547,292	844	7,546,448		7,546,448	[21]	5,543,453		1,847,817	[22] 155,178	4	5	Nov. 27	184,781,753
Maine	4,347,071	237,575	4,109,496	[23] 59,394	4,168,890	28,847	2,070,022	2,070,021			4	4		[24] 102,737,416
Maryland	7,285,117	293,929	6,991,188		6,991,188	9,000	5,525,750			[25] 1,456,438	4	4		174,779,706
Massachusetts	10,721,664	158,717	10,562,947		10,562,947	20,000	7,407,323	2,500,000	635,624		2	2		528,147,350
Michigan	23,783,308	2,109,429	21,673,879	39,610	21,713,489	41,373	1,508,626	6,623,880	3,000,000	[26] 539,610	3	3		722,462,626
Minnesota	11,432,686	1,073,575	10,359,111		10,359,111		6,906,074	3,453,037			3	3		345,303,709
Mississippi	6,791,177		6,791,177	[27] 126,398	6,917,575	6,645	2,848,126	3,855,364		[28] 207,440	5	5		135,823,574
Missouri	8,901,169	262,008	8,639,161		8,639,161	56,760	8,582,401				2	2		431,958,060
Montana	3,873,713	931,834	2,941,879		2,941,879	13,500	2,928,379				5	5		58,837,575
Nebraska	9,149,484	89,062	9,060,422		9,060,422	7,500	6,789,691	2,263,231			4	4		226,510,543
Nevada	744,615	69,603	675,012		675,012	[29]	675,012				4	4		16,875,292
New Hampshire	2,589,833	90,355	2,499,478		2,499,478		1,874,608		624,870		4	4		62,486,940
New Jersey		[30]	11,342,896	37,335	11,380,231	18,660	11,268,571			[31] 93,000	2	3	Dec. 1	546,685,108
New Mexico	2,719,281		2,719,281	42,606	2,761,887	55,238	1,842,649		864,000		5	5		54,385,614
New York	29,536,436	[32] 1,060,146	28,476,290		28,476,290	[33]	21,319,718	5,685,258		[34] 1,471,314	2	2		[35] 1,438,582,716
North Carolina	13,174,064	640,610	12,533,454		12,533,454	[36]	8,845,113		3,688,341		5	5		250,669,089
North Dakota	3,405,212	1,435,908	1,969,304	2,682	1,971,986	25,000	1,290,000	645,000		[37] 11,986	3	3		65,643,460
Ohio	39,023,304	1,941,853	37,081,451		37,081,451		23,175,907	7,416,290		[18] 6,489,254	4	4		927,036,272
Oklahoma	12,924,521	832,101	12,092,420		12,092,420	62,684	9,022,302	3,007,434			4	4		302,310,488
Oregon	6,787,295	588,518	6,198,777		6,198,777	14,967	6,183,810				4	4	1/1	[38] 154,986,497
Pennsylvania	33,482,609	166,880	33,315,729	[39] 307,781	33,623,510	[40] 278,875	25,250,576	4,644,213	3,449,846		4	3	7/1	928,842,534
Rhode Island	1,776,773	44,514	1,732,259	3,488	1,735,747		1,301,810		433,937		2	2		86,612,980
South Carolina	7,162,945	18,635	7,144,310	1,401	7,145,711		3,475,756	1,190,952	[41] 2,479,003		6	6		119,071,835
South Dakota	5,149,295	1,645,413	3,503,882		3,503,882	12,750	2,715,155		775,977		4	4		87,597,064
Tennessee	10,719,195		10,719,195		10,719,195	53,596	6,399,359	2,133,120	[12] 2,133,120		5	5		214,383,900
Texas	32,341,499	2,814,401	29,527,098		29,527,098		22,145,324			[42] 7,381,774	4	4		738,177,457
Utah		[43]	2,104,823	706	2,105,529	3,836	1,663,193		438,500		3½	3½		60,137,811
Vermont	1,879,921		1,879,921		1,879,921	[44]	1,879,921				4	4		46,998,012
Virginia	11,426,066	651,008	10,775,058		10,775,058	[45]	7,542,541	3,232,517			5	5		215,501,157
Washington	7,951,587	698,338	7,253,249		7,253,249	[46]	4,835,499	2,417,750			3	3		241,774,964
West Virginia	5,615,926	257,298	5,358,628	8,450	5,367,078		2,686,823		2,680,255		4	4		133,965,701
Wisconsin	8,757,555	442,714	8,314,841		8,314,841	10,900	3,057,771	4,647,465		[18] 598,705	2	2		415,742,027
Wyoming	1,447,005		1,447,005		1,447,005		1,085,254	361,751			4	4		36,175,118
District of Columbia	1,610,770	11,081	1,599,689		1,599,689					[47] 1,599,689	2	2		79,984,431
Total			493,865,117	818,293	494,683,410	1,102,187	338,927,564	96,225,637	[48] 31,049,036	[49] 27,378,986	Average 3.35 cents			14,751,308,978

[1] Net gasoline tax earned after deduction of refunds allowed by law.
[2] Many States pay collection costs from other State funds, and amounts reported are noted.
[3] Payments for State highway bonds except as noted.
[4] Paid from State budget, $14,685.
[5] Includes $1,326,274 for county bonds.
[6] Includes $50,227 for State highway commission office expenses, and $21,275 for city streets.
[7] Includes administrative costs of State board of equalization which controls fuel tax and transportation tax
[8] Includes all expenses of State inspector of oils.
[9] For town and city streets.
[10] Exemptions made at time of purchase on 9,349,546 gallons.
[11] Paid from motor vehicle receipts, $30,000.
[12] Payments on county road bonds.
[13] For schools and school buildings, $3,783,949, and for reserve fund, $19,680.
[14] For public Schools.
[15] Aviation gas tax fund, for aviation purposes.
[16] Omits 173,975 gallons sold for aviation purposes.
[17] The 2 per cent allowance for evaporation and loss on gross purchases by retailers not shown here.
[18] For city streets.
[19] Includes $3,000,000 formerly a reserve for refunds on right of way, bridges, and culverts.
[20] Paid from State general fund, $15,000.
[21] Paid from State appropriation, $57,500.
[22] For dock board.
[23] Consists of 1 cent tax on all gasoline not used by motor vehicles (3 cent refund).
[24] Excludes gallonage taxed 1 cent, not used by motor vehicles.
[25] Includes $1,381,438 to Baltimore streets and grade separations; also $75,000 for oyster propagation.
[26] Includes $500,000 for city streets, $35,350 for aviation fund, and $4,260 to State general fund.
[27] Receipts from extra 2 cent tax in Harrison County and 3 cent tax in Hancock County for sea-wall.
[28] Allotted to sea-wall financing (largely from extra tax receipts).
[29] Paid from tax commission fund, $1,500.
[30] Exemption made at time of purchase on 96,681,081 gallons.
[31] Includes $90,000 for free bridge commission and department of navigation and commerce, also $3,000 for public utility commission.
[32] Includes $764,782, refunded and an allowance to distributors of $295,364.
[33] Paid from State appropriation of $65,497.
[34] Includes $50,000 to reserve for refunds and $1,421,314 to New York City funds.
[35] Includes 14,768,218 gallons taxed, but taxes returned to distributors for collection costs.
[36] Paid from State highway commission maintenance fund, $8,686.
[37] Includes $11,334 to reserve for refunds, and $652 (license fees) to State general fund.
[38] Includes approximately 136,579 gallons of distillate taxed 3½ cents per gallon.
[39] Includes $299,591 for previous years' taxes.
[40] Collection bonus to dealers deducted before payment of tax to State.
[41] Payments of $300,000 on State highway bonds and $2,179,003 on county road bonds.
[42] For free school fund.
[43] An allowance of 2 per cent for evaporation and handling exempts 1,227,178 gallons from tax.
[44] Paid from motor vehicle funds, $1,000.
[45] Paid from State appropriations of $15,144.
[46] Paid from motor vehicle fund, $5,000.
[47] For repair and improvement of Washington streets.
[48] Includes payments of $10,179,135 on county bonds, and $20,869,901 on State bonds.
[49] Includes $11,842,930 for city streets, $13,404,200 for schools, and $2,131,856 for various items.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.
Report of the Chief of the Bureau of Public Roads, 1929.
Report of the Chief of the Bureau of Public Roads, 1930.

DEPARTMENT BULLETINS

No. *136D. Highway Bonds. 20c.
*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*660D. Highway Cost Keeping. 10c.
1216D. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
1279D. Rural Highway Mileage, Income, and Expenditures 1921 and 1922.
1486D. Highway Bridge Location.

DEPARTMENT CIRCULARS

No. 331C. Standard Specifications for Corrugated Metal Pipe Culverts.

TECHNICAL BULLETIN

No. 55T. Highway Bridge Surveys.

MISCELLANEOUS CIRCULARS

No. 62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-Aid Highway Projects.
*93M. Direct Production Costs of Broken Stone. 25c.
109M. Federal Legislation and Regulations Relating to the Improvement of Federal-Aid Roads and National Forest Roads and Trails, Flood Relief, and Miscellaneous Matters.

MISCELLANEOUS PUBLICATIONS

No. 76MP. The Results of Physical Tests of Road-Building Rock.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *914Y. Highways and Highway Transportation. 25c.
937Y. Miscellaneous Agricultural Statistics.
1036Y. Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio. (1927)
Report of a Survey of Transportation on the State Highways of Vermont. (1927)
Report of a Survey of Transportation on the State Highways of New Hampshire. (1927)
Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio. (1928)
Report of a Survey of Transportation on the State Highways of Pennsylvania. (1928)

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

*Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID AND EMERGENCY ROAD CONSTRUCTION

AS OF

APRIL 30, 1931

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION						BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	BALANCE OF EMERGENCY ADVANCE FUNDS AVAILABLE FOR NEW PROJECTS
		ESTIMATED TOTAL COST	FEDERAL AID ALLOTTED	EMERGENCY ADVANCE FUND	MILEAGE			ESTIMATED TOTAL COST	FEDERAL AID ALLOTTED	EMERGENCY ADVANCE FUND	MILEAGE				
					INITIAL	* STAGE	TOTAL				INITIAL	STAGE	TOTAL		
		$	$	$				$	$	$				$	$
ALABAMA	2,143.1	6,304,189.02	3,089,850.09	992,086.47	200.2	87.4	287.8	1,355,314.95	676,882.70	678,352.25	36.0	35.6	71.8	3,345,444.87	28,206.28
ARIZONA	869.5	4,898,026.08	3,383,483.05	836,633.18	182.2	176.8	359.0	596,972.77	318,491.96	233,189.25	18.8	42.4	81.2	1,150,275.97	100,643.57
ARKANSAS	1,789.6	7,738,949.88	3,584,532.80	1,088,157.00	207.5	58.7	266.2	1,509,354.70	728,028.23	195,000.00	38.2	22.3	58.5	877,548.70	105,000.00
CALIFORNIA	1,963.6	10,293,715.50	4,353,834.51	1,245,006.40	271.2	39.3	310.5	2,724,850.51	1,044,300.99	840,762.51	44.7	17.9	82.8	1,884,286.08	1,022,484.09
COLORADO	1,291.0	4,572,913.12	2,420,239.55	691,801.62	157.0	89.0	248.0	2,054,705.69	1,123,232.45	537,297.67	89.1	21.4	110.5	2,135,633.62	178,732.71
CONNECTICUT	258.2	4,543,863.45	1,755,735.33	520,491.00	43.9		43.9	484,683.59	120,645.53		5.5		5.5	107,028.70	
DELAWARE	302.4	1,102,292.38	550,391.68	400,000.00	56.2		56.2	81,139.00	40,569.50		3.0		3.0	122,256.88	
FLORIDA	540.0	5,066,324.11	2,396,159.84	521,898.76	132.0		132.0	685,590.17	281,222.78	404,367.39	18.5		18.6	1,985,953.96	150,170.85
GEORGIA	2,755.4	8,374,440.27	4,081,666.28	1,587,033.71	304.4	94.5	398.9	1,741,483.89	835,386.80	353,570.10	47.8	24.3	72.1	1,716,298.30	37,392.19
IDAHO	1,296.9	2,573,027.93	1,515,037.43	246,498.74	175.3	11.7	187.0	1,701,383.25	881,190.89	699,952.02	114.8	24.8	139.7	922,358.35	61,584.24
ILLINOIS	2,196.5	21,855,064.75	9,934,551.88	877,000.00	617.8	57.3	675.1	8,348,294.61	4,268,455.74	2,360,111.72	300.2	4.7	304.9	2,527,670.77	183,004.28
INDIANA	1,579.1	6,246,533.83	3,216,459.72	479,512.51	162.7		162.7	5,563,006.99	2,430,596.62	1,189,068.74	188.1		188.1	1,097,142.51	397,347.75
IOWA	3,177.3	5,989,487.33	2,524,410.89	1,518,369.00	142.7	52.7	195.4	2,051,840.25	831,065.00	598,000.00	24.5	40.8	85.3	4,069.10	
KANSAS	3,117.8	7,327,770.42	3,510,584.25	751,047.91	421.1	25.8	446.9	3,487,053.39	1,628,418.07	425,503.96	173.8	52.9	226.7	848,244.37	885,747.33
KENTUCKY	1,555.6	5,739,096.45	2,461,851.51	511,676.40	206.0	81.1	287.1	4,024,035.81	1,234,349.03	925,517.33	91.2	83.8	175.1	339,382.32	67,821.27
LOUISIANA	1,420.9	7,500,460.04	3,632,125.18	1,037,343.87	255.0	18.8	273.8	1,742,075.81	702,705.82	105,000.00	21.1	1.4	22.5	179,490.68	5,583.13
MAINE	591.3	2,825,951.90	999,545.52	178,467.38	48.9		48.9	2,868,149.89	1,078,930.97	537,331.64	78.5		78.5	791,969.44	
MARYLAND	704.8	283,984.10	78,782.10	115,578.50		3.9	3.9	1,582,502.10	791,251.02	562,173.40	55.4	.4	65.8	209,817.29	
MASSACHUSETTS	711.7	8,401,439.13	2,095,208.23	1,141,460.00	73.6		73.6	1,615,946.93	309,421.80		17.2		17.2	2,260,942.96	
MICHIGAN	1,798.1	9,411,818.32	3,957,290.06	661,000.00	256.6	21.1	277.7	1,020,317.20	445,587.65	228,000.00	35.0	9.7	44.7	3,347,325.47	1,832,382.00
MINNESOTA	4,123.5	3,539,813.19	1,414,910.51	635,000.00	44.5	147.4	191.8	5,545,487.59	2,329,661.41	1,614,993.00	35.4	168.0	203.4	66,623.49	
MISSISSIPPI	1,778.7	2,929,325.94	1,447,935.40	870,283.73	108.0	69.1	177.1	1,291,465.78	630,108.69	490,924.16	62.4	22.8	85.2	3,939,513.29	73,528.11
MISSOURI	2,559.5	7,645,366.06	2,886,098.37	469,653.74	166.4	42.4	208.8	3,073,500.32	1,305,677.14	1,441,033.33	97.9	40.6	138.5	1,356,149.00	816,135.93
MONTANA	1,856.3	10,136,708.24	5,705,452.50	1,233,498.66	803.3	77.7	881.0	2,002,676.54	1,128,161.21	431,200.00	118.0	109.6	225.5	2,468,078.33	7,231.44
NEBRASKA	3,866.6	6,922,801.61	3,231,832.73	891,135.36	231.5	151.1	382.6	2,827,101.19	1,354,433.39	758,698.38	30.6	133.1	163.7	1,726,018.78	58,197.26
NEVADA	1,249.4	1,367,326.09	1,053,793.28	154,966.13	62.6	91.3	153.9	1,497,973.65	867,359.50	590,674.25	11.5	147.4	159.0	806,204.61	293,997.62
NEW HAMPSHIRE	392.0	608,439.04	229,098.40		9.7		9.7	208,447.30	98,355.00	75,000.00	6.5		8.5	470,344.65	325,000.00
NEW JERSEY	555.2	4,566,555.45	1,406,315.02	923,770.59	64.0		64.0	1,704,508.24	350,474.32	180,000.00	10.9		10.9	1,081,497.42	4,036.41
NEW MEXICO	1,966.2	5,855,661.33	3,802,565.82	1,052,226.97	214.5	106.1	320.6	482,095.12	306,646.46	117,130.42	31.3	10.6	41.9	231,268.14	133,930.61
NEW YORK	2,694.9	30,385,970.33	8,406,537.00	2,448,566.00	469.7		469.7	17,102,696.00	6,690,314.00	1,527,000.00	290.9	7.0	297.9	21,062.84	75,000.00
NORTH CAROLINA	1,924.6	6,398,222.39	3,091,145.32	1,192,797.65	205.9	31.8	237.7	2,237,928.69	1,105,805.98	704,515.53	86.0	27.9	113.9	1,676,726.25	29,460.32
NORTH DAKOTA	4,519.6	2,638,746.24	1,360,215.43	410,587.00	352.9	192.2	545.1	1,828,232.18	898,264.86	664,837.09	152.6	327.3	479.9	1,799,745.46	223,107.41
OHIO	2,558.8	12,870,569.49	3,928,628.88	837,225.35	187.3	6.9	194.2	4,876,958.16	1,363,576.85	2,161,312.65	61.4	23.7	85.1	2,861,396.51	
OKLAHOMA	1,974.9	5,280,941.80	2,571,557.49	555,955.15	235.0	83.3	318.3	3,888,410.57	2,122,878.78	1,370,395.84	117.7	54.9	172.6	28,144.99	
OREGON	1,262.3	7,103,184.83	3,972,035.22	634,946.08	221.7	90.5	312.2	672,687.73	373,513.95	173,241.29	41.2		41.2	844,510.88	512,099.64
PENNSYLVANIA	2,658.1	7,709,665.31	2,569,533.37		49.9		49.9	2,720,215.63	1,228,183.52	1,287,072.66	79.4		79.4	3,953,431.45	2,225,870.34
RHODE ISLAND	209.5	2,858,872.53	1,078,494.76	375,000.00	44.2		44.2	242,238.02	121,119.01	25,000.00	3.8		3.8	118,803.85	
SOUTH CAROLINA	1,825.7	6,503,546.65	2,910,046.87	1,114,636.00	84.1	179.3	263.4							126,884.51	
SOUTH DAKOTA	3,748.8	4,802,106.99	2,723,709.78	908,987.85	393.6	121.5	515.1	258,998.58	144,248.79	72,065.15	29.2	29.8	59.1	1,361,783.20	356,920.00
TENNESSEE	1,458.1	3,013,294.18	1,505,323.22	819,748.88	124.6	7.9	132.5	1,768,167.43	855,567.66	825,940.52	85.7	5.5	91.2	2,018,727.20	96,192.60
TEXAS	7,074.5	14,243,803.00	6,363,979.14	2,104,039.18	636.5	177.9	814.4	7,043,830.62	3,367,618.00	2,981,556.54	260.3	144.3	404.8	4,394,816.28	2,484.30
UTAH	897.4	2,017,298.85	1,299,282.44	350,060.07	127.1	53.8	180.9	988,001.62	465,015.55	494,928.82	39.4	82.3	121.7	839,989.22	81,532.11
VERMONT	306.3	996,110.09	408,025.95	248,117.02	23.3		23.3	102,493.27	51,246.63	25,034.78	.4		.4	159,238.45	126,848.20
VIRGINIA	1,628.4	5,599,046.17	2,597,504.39	1,006,955.70	241.0	30.5	271.5	1,080,293.43	418,205.97	462,857.60	28.1	18.3	46.4	801,472.15	45,688.70
WASHINGTON	1,013.1	4,129,428.77	1,845,198.55	540,846.04	131.4	7.7	139.1	1,309,671.27	550,833.75	641,937.52	41.9	14.4	56.3	1,445,443.98	88,149.44
WEST VIRGINIA	758.2	4,461,883.18	1,688,289.31	400,766.85	100.2	12.5	112.7	1,398,672.84	526,528.57	345,799.49	36.6		38.6	511,214.60	128,817.66
WISCONSIN	2,475.3	3,552,575.37	1,577,374.55	323,000.00	90.6	13.0	103.6	4,311,220.81	1,801,937.37	1,530,500.00	98.2	55.4	153.6	978,267.97	138,910.00
WYOMING	1,692.1	3,273,782.97	2,103,688.10	310,650.35	204.2	162.7	366.8	1,187,894.33	618,498.28	499,831.51	103.1	99.4	202.5	522,818.89	218,901.14
HAWAII	46.3	790,777.25	361,304.04		26.6		26.6	746,899.90	280,361.36	375,983.99	17.2		17.2	1,580,221.20	24,016.01
TOTALS	89,347.2	303,382,273.34	134,151,826.93	36,325,483.77	9,568.6	2,674.5	12,243.1	118,437,468.31	51,113,405.55	32,812,673.49	3,355.2	1,904.9	5,260.1	63,823,123.03	10,831,840.94

* THE TERM STAGE CONSTRUCTION REFERS TO ADDITIONAL WORK DONE ON PROJECTS PREVIOUSLY IMPROVED WITH FEDERAL AID. IN GENERAL, SUCH ADDITIONAL WORK CONSISTS OF THE CONSTRUCTION OF A SURFACE OF HIGHER TYPE THAN WAS PROVIDED IN THE INITIAL IMPROVEMENT.

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 12, NO. 4 JUNE, 1931

PAVEMENT FAILURE CAUSED BY SLIDE IN SIDE FILL

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

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SUBGRADE SOIL CONSTANTS, THEIR SIGNIFICANCE, AND THEIR APPLICATION IN PRACTICE

Reported by C. A. HOGENTOGLER, Senior Highway Engineer, A. M. WINTERMYER, Assistant Highway Engineer, and E. A. WILLIS, Assistant Highway Engineer, United States Bureau of Public Roads

PART I: THE PHYSICAL PROPERTIES OF SOILS AND THEIR EFFECT ON SUBGRADE PERFORMANCE [1]

PRACTICAL information on the subject of subgrades is naturally divided into two classes: That which relates to the study of the soil, and that which relates to the practical utilization of the results of soil studies in the design and construction of roads. The former is of interest primarily to the subgrade testing engineer while the latter is of interest to every engineer who is now engaged in, and every engineering student who ever expects to engage in, the construction of works on the earth's surface.

The subgrade testing engineer who makes the studies must be familiar with the detailed procedures for making the subgrade surveys, the simplied subgrade soil tests, and the more elaborate Terzaghi tests. He must understand the complete significance of the various tests and the particular tests to be used for various purposes. The designing engineer requires only a superficial knowledge of the significance of the various tests and the procedures for making them. His special interest lies in the utilization of the test results to increase the stability and permanence of the structures with which he has to deal.

During the past few years the Bureau of Public Roads, by means of published reports, lectures, and exhibits, has made known to the interested public the progress of its subgrade studies. These researches have now reached a point where it is desirable to coordinate and summarize the results obtained.

The present report, of which the first part is published in this issue, consists of three major divisions: (1) A discussion of soil properties important with respect to subgrade performance, (2) the significance of the simplified soil tests for disclosing the presence of the important subgrade soil properties, and (3) the practical utilization of subgrade soil tests in practice.

The first division, which is intended primarily for the designing engineer and the engineering student, is included in Part I. In this part of the report the authors attempt in as simple a manner as possible to disclose the relation between the vehicle, the road, and the subgrade groups which have been suggested in a previous report and to discuss in a consistent order the various physical principles controlling the performance of the subgrade. An effort is made to show (*a*) that the subgrade instead of the pavement really supports the wheel load, (*b*) that the manner in which the subgrade supports the wheel load depends upon its reaction to both load and climatic changes, (*c*) that these reactions depend upon the five basic physical characteristics of soils, to wit, cohesion, internal friction, compressibility, elasticity, and capillarity, (*d*) that these physical characteristics control such important performances of subgrades as shrinkage, expansion, frost heave, the settlement of fills, sliding in cuts and lateral flow of soft undersoils, (*e*) that these physical characteristics are furnished by soil constituents easily identified in the laboratory and (*f*) that subgrades may be arranged in definite groups according to the characteristics of the soil constituents.

IMPORTANCE OF SUBGRADE SOIL CONSTANTS DISCUSSED

A subgrade soil test result may be defined as a measure of the degree in which a particular physical characteristic is exhibited when a soil is tested according to some arbitrary procedure. A subgrade soil constant may be either a test result as such or the result of a computation involving the use of several test results.

The subgrade soil constants to be employed beneficially in practice must serve to disclose the existence of those subgrade properties which exert an important influence upon the service rendered by road surfaces.

In order that subgrade soil constants may perform this service, one must have some conception of (*a*) those physical characteristics of subgrade soils which have an important bearing on the serviceability of road surfaces, (*b*) the influence exerted by the condition in which the soil exists and the character of its constituents upon the important subgrade soil properties, (*c*) the laws which control the physical characteristics possessed by subgrade soils, and (*d*) the degree to which subgrade soil constants disclose the presence of important subgrade characteristics.

Information of the character referred to is furnished by the subgrade investigations and the reports regarding them supplemented which have been published at different times in PUBLIC ROADS and elsewhere. These reports are listed in the bibliography included as part of this report.

While there is no intent to minimize in any manner the important influence exerted upon the properties of the soil by the state in which it exists, this report discusses primarily those properties characteristic of the raw constituents of soils regardless of state, and the importance of those properties with respect to road construction.

It should be remembered that the suggested subgrade groups are based upon subgrade performance. As additional information becomes available it might be desirable to subdivide certain of the groups with respect to the degree in which the subgrades possess particular properties, but the main groups are not likely to change in definition. The test constants which are being suggested as a means of identifying the members of the various groups are in a state of development and can not be considered as final. However, these constants and the scheme suggested for their use constitute the most logical method of soil identification yet disclosed by the bureau's subgrade investigations. This material is presented at this time not as a final and conclusive treatise on soil identification, but rather as a rational method by means of which the usefulness of test constants may be intelligently investigated.

Italic figures in parentheses ( ) used in this report refer to reports listed in the bibliography which furnish the material being discussed.

[1] Parts II and III of this article will appear in the July, 1931, issue of Public Roads.

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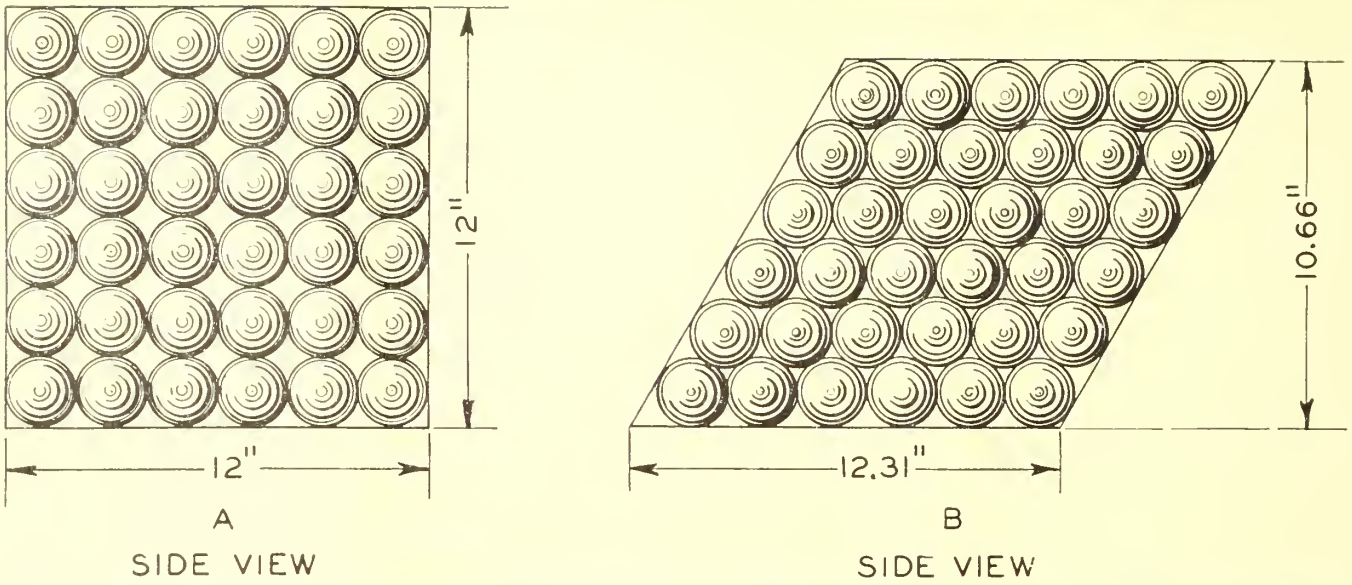


FIGURE 1.—SPHERES 2 INCHES IN DIAMETER ARRANGED IN LOOSE STATE, SIX POINTS OF CONTACT (A) AND IN MORE CONSOLIDATED STATE, EIGHT POINTS OF CONTACT (B)

Procedures for determining the different constants and for making subgrade surveys and mapping subgrade soil profiles are being prepared for publication at a later date.

BOTH SUPPORTING VALUE AND MOVEMENT OF SUBGRADE IMPORTANT IN ROAD CONSTRUCTION

In order to appreciate the importance of the subgrade in road construction one has only to remember that instead of the road surface, the ground or subgrade beneath really supports his pleasure car, truck, or other vehicle. The road surface or pavement merely distributes the vehicle weight over areas larger than those furnished by the tires alone.

Our high-class pavements, even the most substantial, when robbed of this ground support for appreciable distances, say 15 or 18 feet, break of their own weight and when unsupported for even short distances they become unable to withstand the weight of motor cars.

Thus the road surface furnishes only a smooth top on the natural ground surface and in order to accomplish this purpose two conditions must be fulfilled: The road surface must distribute the weight or impact delivered by motor vehicle wheels over an area sufficient to prevent appreciable depression of the ground beneath the pavement, and the ground beneath the pavement must be prevented from moving an amount sufficient to deform the road surface seriously. Otherwise, the road surface will fail.

When designing a highway the engineer is called upon to furnish a structure which, first, will resist the wear and tear caused by fast-moving motor wheels, second, will distribute wheel weights and impacts so as to prevent deformations which would be detrimental to either the road surface or the subgrade and, last, will resist natural forces to such an extent that their effect as manifested through subgrade movement will not be detrimental. In order to do this most economically, he must be cognizant of (1) the wheel loads and impacts to be resisted, (2) the relative ability of pavements to spread or distribute wheel loads, (3) the safe load the subgrade will support without depressing a detrimental amount, and (4) the movements likely to occur in the subgrade due either to climatic influences or to other causes. Only then is the engineer in a position to transform his road appropriations into the greatest mileage of serviceable highways.

It becomes evident, therefore, that in addition to studies of traffic weights and intensities and of pavement properties, none of which are discussed in this report, it is of great importance to investigate both the load-carrying properties of the subgrade and those soil characteristics which control subgrade movements other than those caused by vehicular loads.

VOIDS RATIO, VOLUME CHANGE, MOISTURE CONTENT, AND POROSITY EXPLAINED

It is necessary at this point to define certain terms, with full explanations of their significance. While the engineer who seeks only a general knowledge of subgrades may never have to use them in tests of his own, it is essential to an understanding of the subject that he know the precise meaning of the terms "voids ratio," "volume change," "moisture content," and "porosity," which have to do with those changes in soil state that affect the performance of subgrades.

A soil mass, or soil, as generally termed, consists of both soil particles and pores. When a soil mass, due to change in either moisture content or degree of consolidation, either increases or decreases in volume, only the void volume or the pore space is assumed to change, the volume of the soil particles remaining constant.

The density which controls in a large measure the supporting value of the soil depends upon the ratio of pore volume to either soil particle or soil mass volume. The test constants which represent either the moisture contents of soils when in particular states, or the changes in moisture content caused by changes in soil states, indicate, among other things, the density of the soil.

In order to visualize the soil states indicated by the constants and by the different degrees of soil density one must thoroughly understand the significance of the terms "voids ratio," "moisture content," and "porosity," which disclose the relation of pore volume to soil particle volume in the soil mass.

Voids ratio.—This term is defined as the ratio of the volume of voids to the volume of soil particles in a soil mass (*1*), i. e., the volume of the voids or pores per unit volume of soil particles in a soil mass.

Thus if e = voids ratio;
V_v = volume of voids;
V_s = volume of soil particles;

$$e = \frac{V_v}{V_s} \quad (1)$$

and $e+1$ = total volume of soil mass per unit volume of soil particles in the mass.

The voids ratio, e, varies with (1) variation in degree of compaction, the number of soil particles remaining constant, (2) increase or decrease in moisture content, the number of soil particles remaining constant, and (3) increase or decrease in total number of soil particles, the volume of the soil mass remaining constant.

To illustrate the significance of the voids ratio, assume 216 spheres to be arranged as shown in Figure 1, A in a container 1 cubic foot in volume and a rectangular parallelepiped in form.

The combined volume of the voids and the spheres representing that of the soil mass equals that of the container, 1,728 cubic inches.

The volume of the spheres is given by the equation,

$$V_s = \frac{4}{3}\pi r^3 \times 216 = 904.8 \text{ cubic inches}$$

The volume of voids is the difference between these two volumes,

$$V_v = 1728 - 904.8 = 823.2 \text{ cubic inches}$$

Hence the voids ratio,

$$e = \frac{823.2}{904.8} = 0.910$$

By a rearrangement of the spheres the voids ratio may be changed. This is illustrated by placing the 216 spheres in a container which is an oblique parallelepiped in form, as shown in Figure 1, B.

In this case the volume of the container is equal to the product, $12 \times 12.31 \times 10.66 = 1{,}575$ cubic inches. $V_v = 1{,}575 - 905 = 670$ cubic inches; and the void's ratio, $e = \frac{670}{905} = 0.740$.

Volume change.—This term is defined as the change in the volume of a soil mass due to change in the state of consolidation of the soil particles. Volume change is expressed in percentage of the volume of the soil mass either before or after change in the state of consolidation.

Thus when a soil changes from one state of consolidation indicated by a volume equal to V_1 to a different state of consolidation indicated by a volume V_2, C_1 is the volume change in percentage of the volume of the soil, V_1; and C_2 is the volume change in percentage of the volume V_2. Thus

$$C_1 = \frac{V_1 - V_2}{V_1} \times 100$$

$$C_2 = \frac{V_1 - V_2}{V_2} \times 100$$

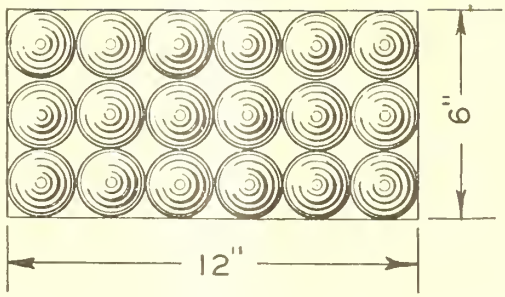


FIGURE 2.—108 SPHERES ARRANGED WITH SIX POINTS OF CONTACT ON EACH SPHERE, VOIDS RATIO EQUAL TO THAT OF THE 216 SPHERES SHOWN IN FIGURE 1, A

If the spheres shown in Figure 1, A are assumed to represent a state of consolidation indicated by V_1 and those shown in Figure 1, B are assumed to represent a state of consolidation indicated by V_2, the volume change obtained by changing the arrangement of the spheres from that shown in Figure 1, A to that shown in Figure 1, B is given by the formula,

$$C_2 = \frac{V_1 - V_2}{V_2} \times 100 = \frac{1{,}728 - 1{,}575}{1{,}575} \times 100 = 9.7 \text{ per cent}$$

This method of computing volume change may be used whenever the volume of the soil mass but not the number of soil particles changes.

When a soil changes from any wet state indicated by a volume equal to V to the thoroughly dried state indicated by the volume equal to V_o, C is defined as the volume change in percentage of the volume of the soil in the wet state and C_o is defined as the volume change in percentage of the volume of the soil in the dry state. Thus

$$C = \frac{V - V_o}{V} \times 100; \text{ and}$$

$$C_o = \frac{V - V_o}{V_o} \times 100 \quad (2)$$

Thus, for instance, the volume change which occurs when soil cakes in the laboratory are changed from the wet to the dry state, as in shrinkage tests, is given by this formula.

In case, however, one desires to compute the volume change when both the degree of consolidation and the number of soil particles change he must of necessity employ the voids ratio, which is expressed in unit values.

Assume, for instance, that the arrangement of the spheres shown in Figure 1, A remains the same but that the number of spheres is reduced to 108 as shown in Figure 2. The volume of the container is reduced by one-half but the voids ratio of necessity remains the same,

$$e = \frac{864 - 452.4}{452.4} = 0.910$$

Let e_1 equal the voids ratio possessed by the 108 spheres shown in Figure 2, state 1, and e_2 equal the voids ratio of the 216 spheres shown in Figure 1, B, state 2. Then, if the degree of consolidation of the spheres is changed from state 1 (figs. 1, A or 2) to state

2 (fig. 1, B), the volume change of the container or soil mass per unit volume of soil particle, expressed in percentage of the volume of the container per unit volume of soil particle, state 2, is given by the expression,

$$C_2=\frac{(1+e_1)-(1+e_2)}{1+e_2}\times 100$$

$$=\frac{e_1-e_2}{1+e_2}\times 100 \quad (3)$$

$$=\frac{0.910-0.740}{1+0.740}\times 100=9.8 \text{ per cent}$$

which substantially agrees with the value of C_2 obtained above, the difference being due to the lack of decimal places in the values of e.

Moisture content.—The moisture content, w, is defined as the weight of moisture in the soil in percentage of the weight of the soil particles.

Thus, if M_w is defined as the weight of the soil moisture in grams and W_o as the weight of the soil particles (weight of thoroughly dry sample) in grams,

$$w=\frac{M_w}{W_o}\times 100 \quad (4)$$

To determine the moisture content possessed by a soil, the soil sample is weighed first wet and then dry. Hence, if W is defined as the weight of the wet sample (weight of soil particles + weight of moisture) and W_o as the weight of the dried soil sample,

$$M_w=W-W_o \quad (5)$$

and

$$w=\frac{W-W_o}{W_o}\times 100 \quad (6)$$

Because of the fact that 1 cubic centimeter of water weighs 1 gram, the weight of the water in grams, M_w, is also the volume of water in cubic centimeters. The volume of the soil particles in cubic centimeters equals the weight of the soil particles in grams divided by the specific gravity of the soil particles. Thus, if

V_s = volume of the soil particles in cubic centimeters;

and

G = specific gravity of the soil particles; then

$$V_s=\frac{W_o}{G} \quad (7)$$

Consequently, w_v, the moisture content of the soil in percentage of the volume of the soil particles is given by the equation,

$$w_v=\frac{M_w}{\frac{W_o}{G}}\times 100$$

$$=\frac{W-W_o}{W_o}\times 100\times G$$

$$=wG \quad (8)$$

Since w_v, the moisture content in percentage of the volume of the soil particles is equal to the void volume in percentage of the volume of soil particles, when the voids are completely filled with water, we have

$$w_v=e\times 100 \quad (9)$$

and by substitution of wG for w_v,

$$e=\frac{wG}{100} \quad (10)$$

Thus, if a soil sample weighs 30 grams when wet, 25 grams when dry, and the soil particles have a specific gravity of 2.5,

$$M_w=W-W_o=30-25=5 \text{ grams};$$

$$w=\frac{M_w}{W_o}\times 100=\frac{5\times 100}{25}=20 \text{ per cent};$$

$$w_v=w\times G=20\times 2.5=50 \text{ per cent};$$

$$e=\frac{w_v}{100}=0.5.$$

Porosity.—The porosity, P, is defined as the volume of the voids or pores in a soil mass in percentage of the volume of the soil mass (volume of soil particles + volume of the voids). Its value is given by the formula,

$$P=\frac{V_v}{V_v+V_s}\times 100=\frac{e}{1+e}\times 100 \quad (11)$$

Thus, for the soil sample referred to above,

$$P=\frac{e}{1+e}\times 100=\frac{0.5\times 100}{1+0.5}=33.3 \text{ per cent.}$$

For the spheres shown in Figure 1, A,

$$P=\frac{0.91}{1+0.91}\times 100=47.6 \text{ per cent}$$

and for the spheres shown in Figure 1, B,

$$P=\frac{0.740}{1+0.740}\times 100=\frac{74}{1.74}=42.5 \text{ per cent.}$$

It has been shown [2] that the porosity of spheres of equal size when in the densest possible state equals approximately 26 per cent and the corresponding voids ratio approximately 0.35.

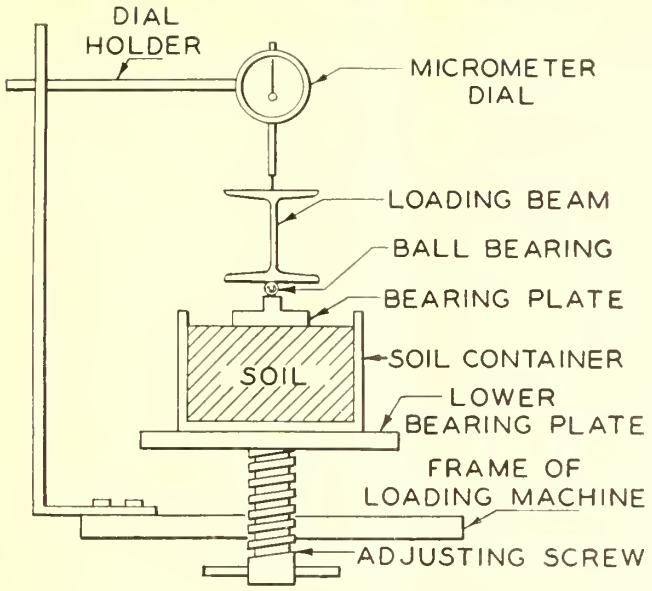
INFORMATION FURNISHED BY SUPPORTING VALUE TESTS LIMITED IN SCOPE

A scrutiny of reports on experiments dealing with the subject discloses a diversity of opinion as to the best manner not only of measuring but also of expressing the magnitude or efficiency of the support furnished to the road by the subgrade.

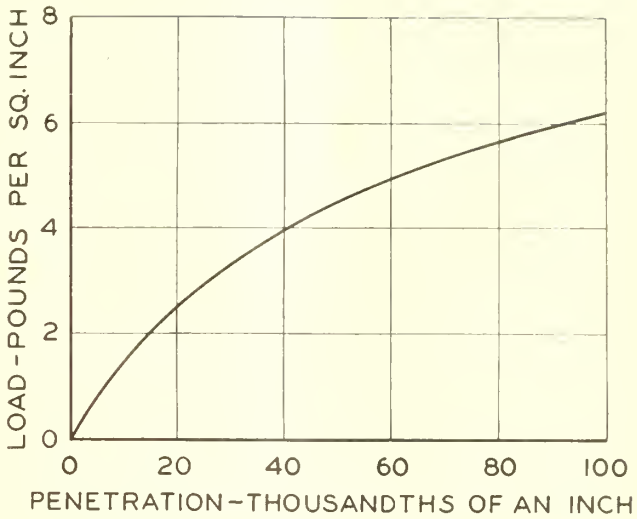
Thus the "comparative bearing value" of soils studied in the early investigations of the Bureau of Public Roads (*2*), the "modulus of subgrade reaction" used by H. M. Westergaard (*3*) in his discussions, and the "consistency" of soils investigated by Charles Terzaghi (*1, 4*) are all indicative of supporting value and yet differ widely in both significance and scope.

The nature of the comparative bearing value test and the type of information furnished by it are illustrated in Figure 3. It is, in brief, a simple load-deformation, or load-penetration test of a small soil sample held in a container. A curve is drawn which shows

[2] Taylor and Thompson, Concrete, Plain and Reinforced, second edition, 1912, pp. 168-170.



DEVICE FOR MEASURING BEARING VALUE



RESULT OF BEARING VALUE TEST

FIGURE 3.—APPARATUS AND TYPE OF RESULTS FURNISHED BY BEARING VALUE TEST

how much a given load will deform a given sample. Thus, according to Figure 3, a load of 4 pounds per square inch causes the bearing plate to penetrate a particular soil sample 0.041 inch. For comparative purposes, the bearing value of the soil was assumed to be the load in pounds per square inch required to produce a penetration of 0.1 inch.

Doctor Westergaard's modulus of subgrade reaction may be defined as the load in pounds per square inch required to deform a perfectly elastic subgrade 1 inch. Thus a perfectly uniform and perfectly elastic subgrade which will deform 0.01 inch for each one-half pound per square inch of pressure applied, has a modulus of subgrade reaction equal to 50.

The Terzaghi consisteney test is performed on a cylindrical soil sample in the following manner. The sample is mounted in a loading machine equipped with a micrometer dial for measuring deformations, as shown in Figure 4. Load is applied slowly to a predetermined magnitude, and the sample is allowed to deform under this load until a state of equilibrium is reached. During this period of constant load the deformation is recorded as a function of the time. The load is then removed, and applied again to a greater magnitude. The deformation as a function of time is recorded for this load; the load is removed and applied a third time, until a point is reached (the yield point) where deformation is continuous without increase in load. The eurve in Figure 4, center, shows the type of load-deformation curve which results from such a test. The eurve at the bottom shows deformation plotted as a function of time.

It is evident that this test takes into consideration not only the load-deformation relation which the comparative bearing value test was designed to give, and the elastic rebound assumed by Westergaard, but also the effeet of time on the deformation.

Even the consistency test, however, fails to supply complete information on subgrade support which should also include a knowledge of:

1. The deformation of the soil as influenced by (*a*) the magnitude of applied load, (*b*) the size and shape of the loaded area, and (*c*) a surcharge adjacent to the loaded area.

2. The relative amounts of the deformation due to (*a*) lateral displacement of the loaded soil and (*b*) compression of the under soil without lateral displacement.

3. The tendency of the soil to remain compressed or rebound upon the removal of load.

Direct bearing value tests, both in time and effort required, are generally too elaborate for use as routine tests for subgrade soils. Consequently, instead of direct tests of supporting value comparatively simple tests are used in the subgrade investigations to diselose the presence of subgrade characteristics indicative of three properties which either singly or in combination control the many types of deflection produced in soils by loading.

These properties af soils may be defined as follows:

1. Stability, the property of resisting lateral flow when loaded.

2. Compressibility, the property of compressing vertieally under load without lateral movement and with a proportional decrease in air or moisture content.

3. Elasticity, the property of deforming under load and rebounding upon the removal of load without changing mositure content.

Figure 5, top, illustrates the character of deformation produced by loss of stability in soils. Here the load displaees the soil laterally. The deformation due to the compressibility of soils, illustrated in Figure 5, center, consists entirely of a more or less permanent consolidation of soil particles in the vertical direction. Figure 5, bottom, illustrates the rebound upon the removal of load in elastie soils.

Loss of stability may eause fills to slide, clay to work up into the interstiees of base courses and rutting to oceur in flexible road surfaces. Examples of loss of stability are illustrated in Figures 6 and 7.

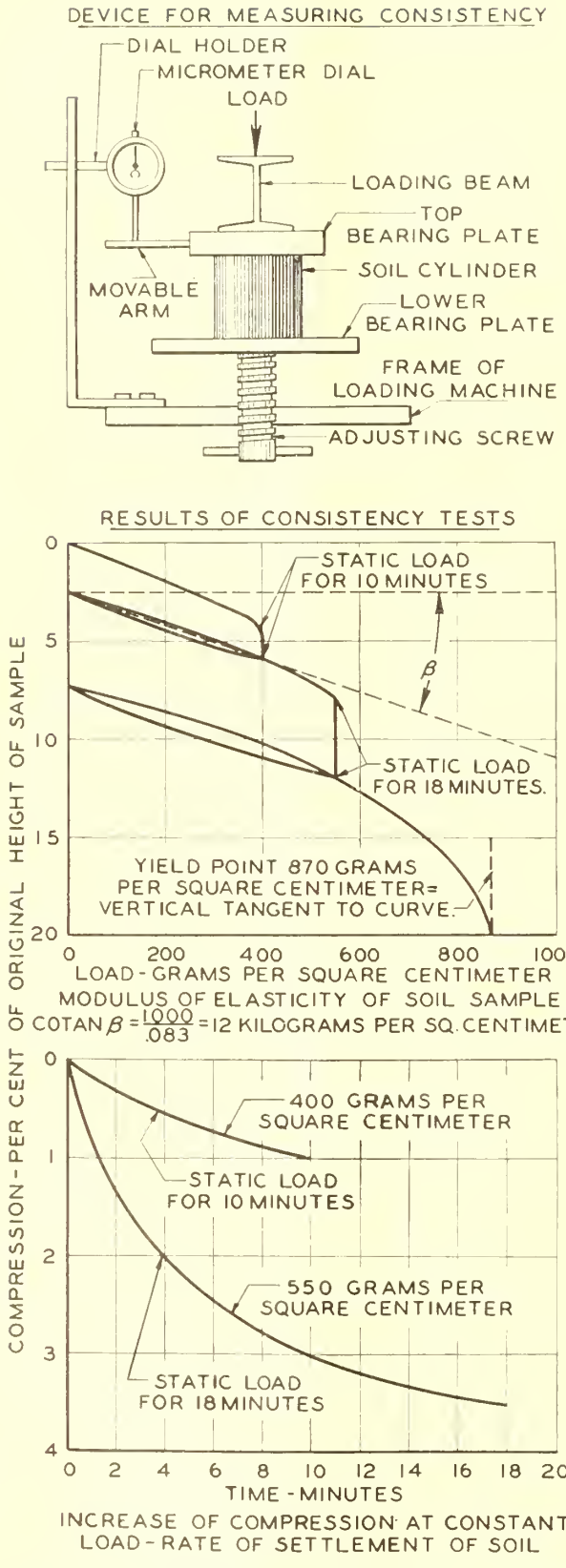

DEVICE FOR MEASURING CONSISTENCY
DIAL HOLDER
MICROMETER DIAL
LOAD
LOADING BEAM
TOP BEARING PLATE
SOIL CYLINDER
MOVABLE ARM
LOWER BEARING PLATE
FRAME OF LOADING MACHINE
ADJUSTING SCREW
RESULTS OF CONSISTENCY TESTS
STATIC LOAD FOR 10 MINUTES
STATIC LOAD FOR 18 MINUTES.
YIELD POINT 870 GRAMS PER SQUARE CENTIMETER= VERTICAL TANGENT TO CURVE.
COMPRESSION - PER CENT OF ORIGINAL HEIGHT OF SAMPLE
LOAD - GRAMS PER SQUARE CENTIMETER
MODULUS OF ELASTICITY OF SOIL SAMPLE
COTAN β = 1000/.083 = 12 KILOGRAMS PER SQ. CENTIMETER
400 GRAMS PER SQUARE CENTIMETER
STATIC LOAD FOR 10 MINUTES
550 GRAMS PER SQUARE CENTIMETER
STATIC LOAD FOR 18 MINUTES
TIME - MINUTES
INCREASE OF COMPRESSION AT CONSTANT LOAD - RATE OF SETTLEMENT OF SOIL

FIGURE 4.—APPARATUS AND TYPE OF RESULTS FURNISHED BY CONSISTENCY TESTS

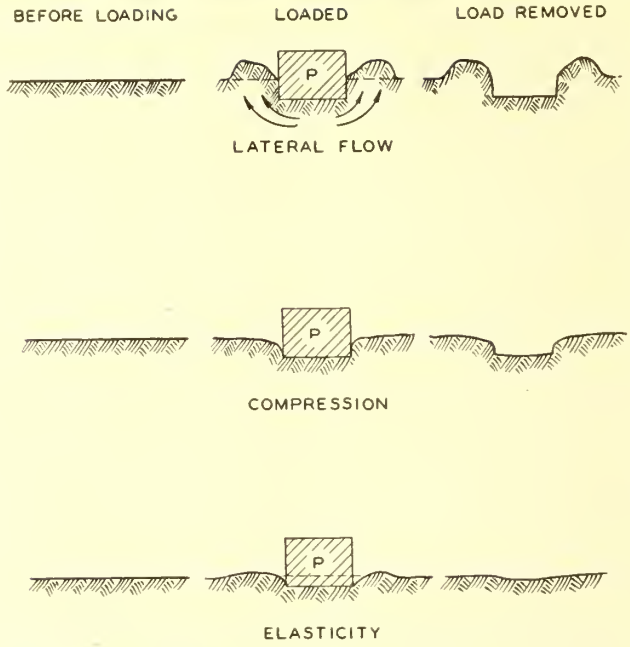


FIGURE 5.—DIAGRAM ILLUSTRATING PROPERTIES ON WHICH REACTION BETWEEN SOIL AND LOAD DEPEND

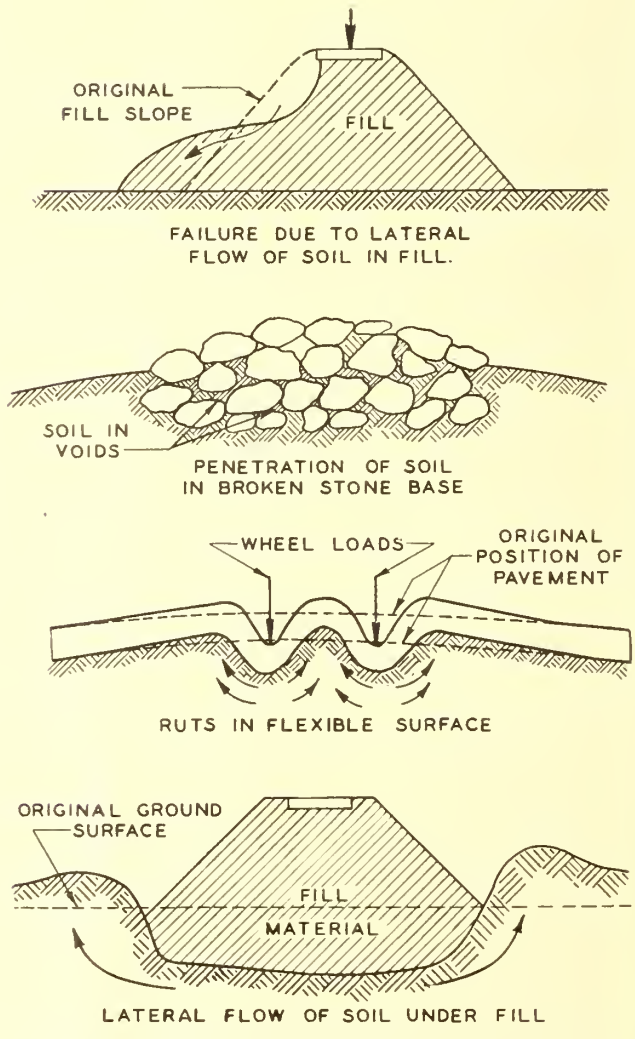

ORIGINAL FILL SLOPE
FILL
FAILURE DUE TO LATERAL FLOW OF SOIL IN FILL.
SOIL IN VOIDS
PENETRATION OF SOIL IN BROKEN STONE BASE
WHEEL LOADS
ORIGINAL POSITION OF PAVEMENT
RUTS IN FLEXIBLE SURFACE
ORIGINAL GROUND SURFACE
FILL MATERIAL
LATERAL FLOW OF SOIL UNDER FILL

FIGURE 6.—TYPES OF ROAD FAILURE CAUSED BY LATERAL FLOW OF THE SUBGRADE SOIL

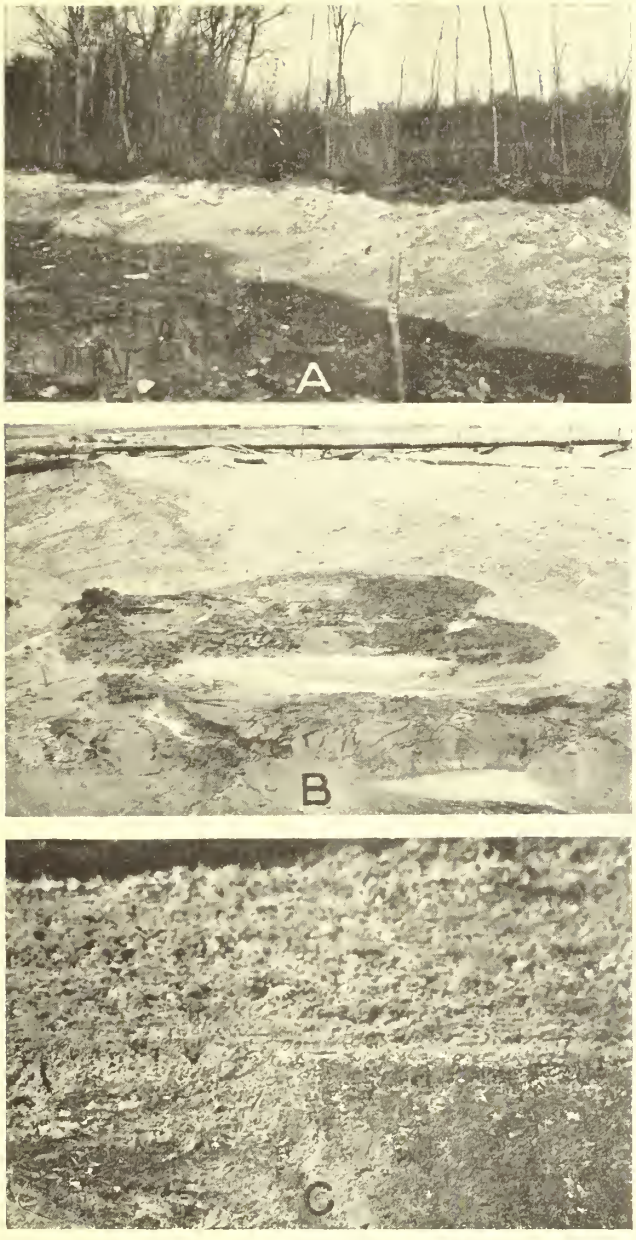


FIGURE 7.—EFFECTS OF LATERAL FLOW IN SOILS. A.—RIDGE OF MUCK SOIL FORCED UP BY LATERAL FLOW OF SOFT UNDERSOIL DUE TO WEIGHT OF FILL MATERIAL. DITCH OPENED AFTER UPHEAVAL. B.—RIVER BOTTOM FORCED UPWARD DUE TO LATERAL FLOW UNDER HYDRAULIC FILL. C.—SLIDE IN FACE OF CUT

STABILITY OF SOILS CONTROLLED BY THE COMBINED EFFECT OF INTERNAL FRICTION AND COHESION

Stability depends upon the shear strength which in turn depends upon the combined effect of the two mechanical properties of soils, internal friction and cohesion.

The magnitude of the cohesion possessed by a soil is independent of the outside pressure acting on the soil. It depends upon the stickiness of the soil grains or their resistance to being pulled apart and thus consists of the true cohesion of soil particles combined with that furnished by the molecular attraction of water (*5, 6*). The stickiness of the clay in sand-clay roads and that of bituminous materials in black-top pavements represent true cohesion of materials. The very stable support furnished racing automobiles by beach sands when wet compared with the low stability of similar sands when dry serves to illustrate the importance of that portion of the total cohesion furnished by the molecular attration of water.

Internal friction, the magnitude of which increases in direct proportion to the pressure exerted upon the soil, depends upon the resistance of the soil grains to sliding over each other (*6*). It is defined as the angle whose tangent is the ratio between the resistance offered to sliding along any plane in the soil and the component of the applied force acting normal to that plane. The sand in sand-clay roads and the mineral aggregate in bituminous surfaces furnish the internal friction.

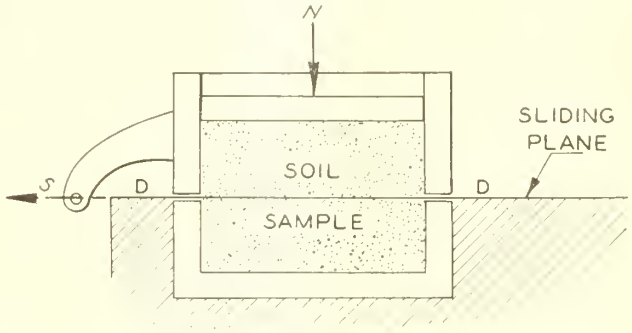


FIGURE 8.—SHEAR RESISTANCE OF SOILS AS RELATED TO INTERNAL FRICTION AND COHESION. BASIC PRINCIPLE OF APPARATUS TO DETERMINE MAGNITUDE OF THESE TWO MECHANICAL PROPERTIES

Figure 8 illustrates the influence exerted by both the cohesion and the internal friction upon the shear strength of soils.

The shear resistance is represented by the vector S. Let

N = Pressure acting on soil sample normal to the sliding plane;
c = Cohesion;
= S, when $N = o$;
ϕ = Angle of internal friction.

Then

$N \tan \phi$ = Frictional resistance to sliding;
$S = N \tan \phi + c$;

and

$$\phi = \text{arc tan } \frac{S - c}{N}.$$

Figure 9 illustrates the conditions required for stability in homogeneous soils.

Let DD be any plane in the soil making the angle α with the horizontal,

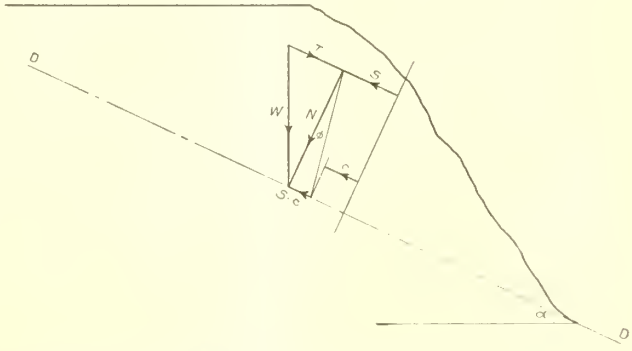


FIGURE 9.—THE MECHANICS OF SLIDING IN HOMOGENEOUS SOILS

W = unit weight of soil;
N = component of W normal to DD;
$= W \cos \alpha$;
then
T = force productive of sliding;
$= W \sin \alpha$;
and
S = shear resistance of soil;
$= N \tan \phi + c$.

Sliding occurs when T exceeds S. Therefore the requirement for stability is that $W \sin \alpha$ be less than $W \cos \alpha \tan \phi + c$.

Based upon this theoretical conception of stability, formulas have been developed by means of which may be determined (*a*) the influence of cohesion, internal friction, width of loaded area, and load adjacent to the loaded area upon the stability of subgrades (*7*) and (*b*) the influence of cohesion, internal friction and slope upon the critical height of fills (*8*).

The formula for computing the supporting value of soils was derived on the assumption that the loaded area was very long compared with its width. This assumption does not satisfy the condition produced by a wheel load upon the pavement, which is not susceptible of simple mathematical treatment. The analysis based upon a long loaded area illustrates the relative influence exerted by cohesion and internal friction upon the stability of soils.

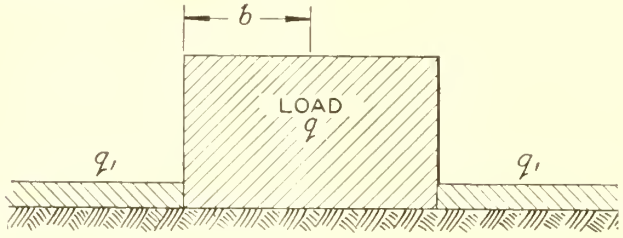


FIGURE 10.—DIAGRAM FOR ANALYSIS OF SUPPORTING POWER OF SOILS

Space does not permit derivation of the formula in this article (*7*). Figure 10 illustrates the method employed.

Let
q = load per unit area;
q_1 = surcharge adjacent to loaded area in same units;
b = ½ width of loaded area;
s = unit weight of soil;
c = cohesion (force per unit area);
ϕ = angle of internal friction;
$\beta = 45° - \frac{\phi}{2}$.

The load q, defined as the supporting value of the soil under the given conditions of cohesion, internal friction, width of loaded area, and surcharge, is given by the formula

$$q = \frac{q_1}{\tan^4 \beta} + \frac{bs}{2 \tan \beta}\left[\frac{1}{\tan^4 \beta} - 1\right] + \frac{2c}{\tan \beta \sin^2 \beta} \quad (12)$$

Résal's formula for computing the critical heights of cuts or fills (see fig. 11) gives only approximate results. It may be stated as follows:

Let
i = angle of inclination of fill;
ϕ = angle of internal friction;
Δ = unit weight of soil;
c = cohesion;

Then h_1, the critical height of fill above which sliding will occur is given by the formula,

$$h_1 = \frac{c \sin i \cos \phi}{\Delta \sin^2 \frac{i - \phi}{2}} \quad (13)$$

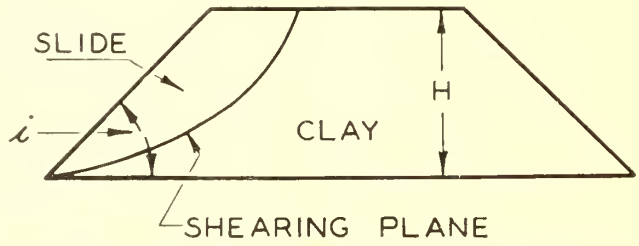


FIGURE 11.—DIAGRAM ILLUSTRATING RÉSAL'S FORMULA FOR COMPUTING THE CRITICAL HEIGHT OF CUTS OR FILLS

Results of computations made according to the formula of Figure 10 are shown in Table 1, and those made according to Résal's formula, Figure 11, are shown in Table 2.

TABLE 1.—*Influence of cohesion, internal friction, width of loaded area and load adjacent to loaded area upon stability of subgrades*

Soil type	Cohesion, c	Angle of internal friction, ϕ	Supporting value, q, pounds per square foot		
			$q_1=0$ $s=100$ pounds per cubic foot $b=0.71$-foot	$q_1=100$ $s=100$ pounds per cubic foot $b=0.71$-foot	$q_1=0$ $s=100$ pounds per cubic foot $b=7.10$ feet
	Pounds per square foot	°			
Clay, almost liquid	100	0	400	500	400
Clay, very soft	200	2	860	980	910
Clay, soft	400	4	1,860	1,990	1,960
Clay, fairly stiff	1,000	6	4,980	5,130	5,170
Clay, very stiff	2,000	12	12,540	12,770	13,060
Silts,[1] wet	0	10	40	240	430
Sands, dry	0	34	770	2,020	7,680
Sand-gravel mixtures, cemented [2]	1,000	34	17,840	19,090	24,750

[1] In silty soils the angle of internal friction may vary between 10° and 30° but the cohesion may be almost 0.
[2] In properly graded soils, depending upon the extent of their compaction, the angle of internal friction may exceed 34° but the cohesion may be considerably less than 1,000.

TABLE 2.—*Critical heights of slope in cuts and fills, computed from Résal's formula*

Soil type	Slope of cut or fill	Angle of slope, i	Weight of soil, Δ	Cohesion of soil,[1] c	Angle of internal friction, ϕ	Critical height of fill, h_1
		° ′	*Pounds per cubic foot*	*Pounds per square foot*	*Degrees*	*Feet*
Very soft clay	½:1	63 26	80	200	2	9
	1:1	45 00				13
	2:1	26 34				25
	4:1	14 02				55
Medium clay	½:1	63 26	90	1,000	6	43
	1:1	45 00				70
	2:1	26 34				155
	4:1	14 02				546
Stiff clay	½:1	53 26	100	1,500	8	61
	1:1	45 00				104
	2:1	26 34				255
	4:1	14 02				1,300
Good sand clays	½:1	63 26	110	1,000	34	104
	1:1	45 00				580
	2:1	26 34				Unlimited.
	4:1	14 02				Unlimited.
Silty clays	½:1	63 26	100	200	14	10
	1:1	45 00				20
	2:1	26 34				72
	4:1	14 02				Unlimited

[1] Materials having no cohesion such as sands, silts, etc., have no critical heights.

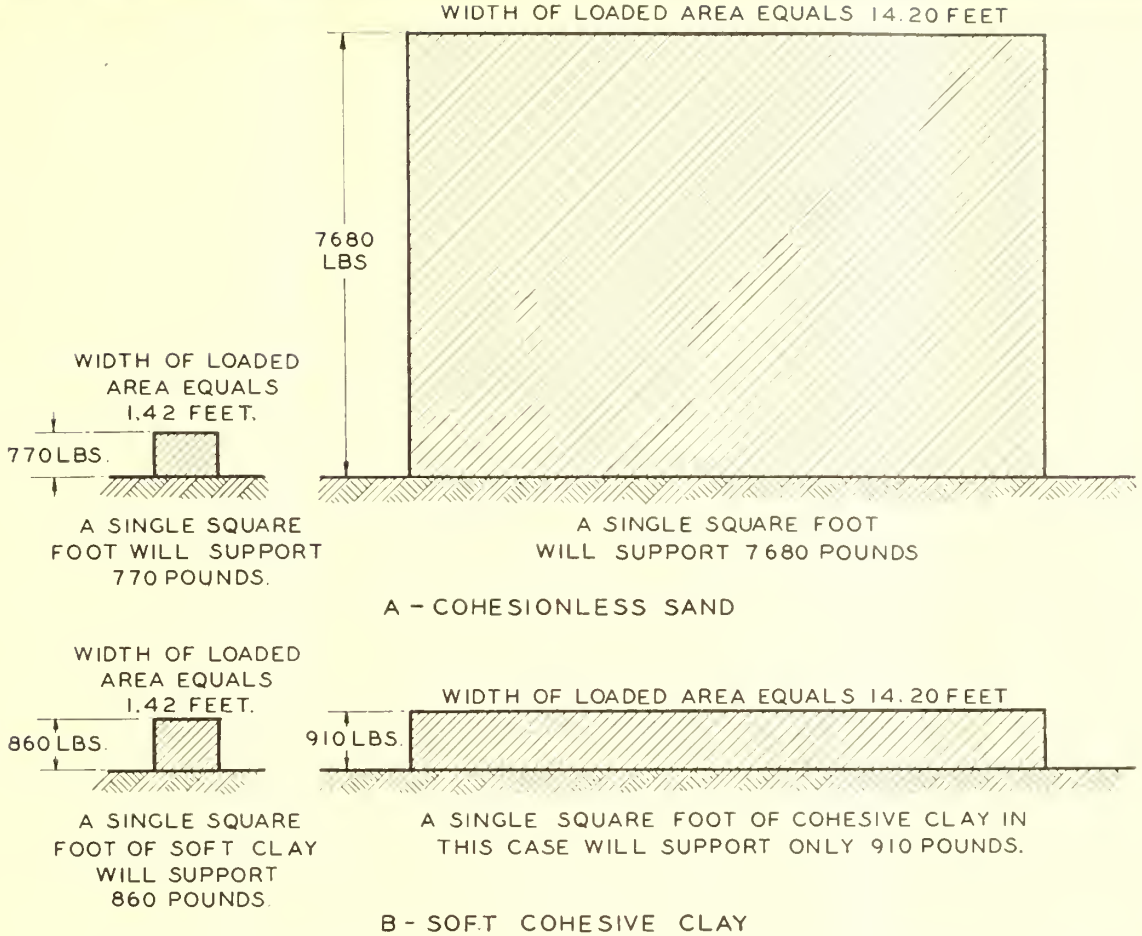

WIDTH OF LOADED AREA EQUALS 14.20 FEET
7680 LBS
WIDTH OF LOADED AREA EQUALS 1.42 FEET.
770 LBS.
A SINGLE SQUARE FOOT WILL SUPPORT 770 POUNDS.
A SINGLE SQUARE FOOT WILL SUPPORT 7680 POUNDS
A - COHESIONLESS SAND
WIDTH OF LOADED AREA EQUALS 1.42 FEET.
860 LBS.
WIDTH OF LOADED AREA EQUALS 14.20 FEET
910 LBS.
A SINGLE SQUARE FOOT OF SOFT CLAY WILL SUPPORT 860 POUNDS.
A SINGLE SQUARE FOOT OF COHESIVE CLAY IN THIS CASE WILL SUPPORT ONLY 910 POUNDS.
B - SOFT COHESIVE CLAY

FIGURE 12.—RELATIVE EFFECT ON SUPPORTING POWER OF SOILS OF INCREASING THE AREA UNDER LOAD

These two formulas also serve to furnish some indication of the relative influence exerted on both the stability of subgrades and the critical height of fills by variations in the angle of internal friction, the cohesion remaining constant, or vice veras. Thus, when the cohesion equals 400 pounds per square foot, b equals 0.71 foot, and q_1 equals 0, the supporting value of the subgrade will equal either 2,550 or 7,600 pounds per square foot, depending on whether the angle of internal friction equals 12° or 34°. When the angle of internal friction equals 34° the supporting value of the subgrade will equal either 4,180 or 17,840 pounds per square foot, depending on whether the cohesion equals 200 or 1,000 pounds per square foot. Likewise, the critical height of fills with a slope of 1 to 1, a weight of soil, Δ, of 100 pounds per cubic foot, and a cohesion of 400 pounds per square foot will be either 21 or 34 feet, depending on whether the angle of internal friction equals 2° or 12°; and when the angle of internal friction equals 6° the critical height will equal 13 or 63 feet, depending on whether the cohesion equals 200 or 1,000 pounds per square foot.

These examples show not only that stability depends upon both the internal friction and the cohesion of the soil, but also that the manner in which stability is influenced by such factors as the size of the loaded area, etc., differs widely depending on whether the stability is furnished principally by internal friction or cohesion. Thus:

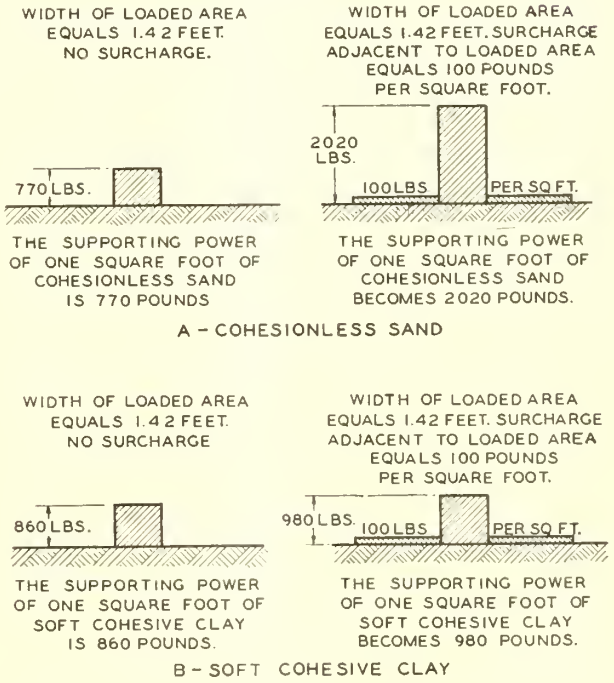

WIDTH OF LOADED AREA EQUALS 1.42 FEET. NO SURCHARGE.
WIDTH OF LOADED AREA EQUALS 1.42 FEET. SURCHARGE ADJACENT TO LOADED AREA EQUALS 100 POUNDS PER SQUARE FOOT.
2020 LBS.
770 LBS.
100 LBS
PER SQ FT.
THE SUPPORTING POWER OF ONE SQUARE FOOT OF COHESIONLESS SAND IS 770 POUNDS
THE SUPPORTING POWER OF ONE SQUARE FOOT OF COHESIONLESS SAND BECOMES 2020 POUNDS.
A - COHESIONLESS SAND
WIDTH OF LOADED AREA EQUALS 1.42 FEET. NO SURCHARGE
WIDTH OF LOADED AREA EQUALS 1.42 FEET. SURCHARGE ADJACENT TO LOADED AREA EQUALS 100 POUNDS PER SQUARE FOOT.
860 LBS.
980 LBS.
100 LBS
PER SQ FT.
THE SUPPORTING POWER OF ONE SQUARE FOOT OF SOFT COHESIVE CLAY IS 860 POUNDS.
THE SUPPORTING POWER OF ONE SQUARE FOOT OF SOFT COHESIVE CLAY BECOMES 980 POUNDS.
B - SOFT COHESIVE CLAY

FIGURE 13.—RELATIVE EFFECT ON SUPPORTING POWER OF SOILS OF UNIFORMLY LOADING THE SOIL ADJACENT TO THE LOADED AREA

1. Increasing the width of the loaded area and also surcharging the soil with a load adjacent to the loaded area increases the unit support very appreciably, as shown in Table 1 and Figures 12 and 13, when the stability of the subgrades is furnished principally by internal friction instead of cohesion. The increases in the unit support of cohesionless soils due to increase in width of loaded area and to the surcharging just noted are, according to unpublished data furnished by Dr. Charles Terzaghi, much greater when the loaded area is long and narrow than when it is square or circular.

2. Increasing the width of the loaded area or surcharging the soil with a load adjacent to the loaded area does not increase the unit support very appreciably when the stability of the subgrade is furnished principally by cohesion instead of internal friction.

3. The safe angle of repose of fill material is independent of the height of the fill only when the fill consists of cohesionless materials.

Thus one sees how a conception of the effect of the relative amounts of cohesion and internal friction possessed by the soil is more enlightening with respect to the design of preventive measures than merely a knowledge of stability or the combined effect of these two properties. Figure 14 shows a method of preventing sliding by terracing the faces of a cut made in clay.

FIGURE 14.—TERRACING OF FACES OF RAILWAY CUT IN CLAY, BETWEEN WASHINGTON AND BALTIMORE. SURFACE DRAINAGE IS PROVIDED FOR ON EACH TERRACE. ONE OF THE METHODS USED TO PREVENT SLIDING IN CUTS

RECOGNITION OF COMPRESSIBILITY AND ELASTICITY IN SUBGRADES IMPORTANT

Consolidating the subgrade serves to increase its density and decrease its permeability (*1, 7, 9, 10*)[3] and consequently is likely to prove highly beneficial. The degree of consolidation obtainable and the nature of the results, whether beneficial or detrimental, which will be obtained by attempted consolidation depend upon whether the subgrade soils are of the compressible or of the elastic type; that is, whether in the absence of change in moisture content they will remain consolidated or will rebound upon the removal of load. To remain consolidated after the removal of load, in the absence of free water, soil grains must either lack any spongy or elastic property which tends to push them apart or possess cohesion in amount sufficient to overcome such a tendency.

A small sponge and a wad of cotton will serve to illustrate very effectively how the elastic subgrades differ in performance from the compressible subgrades. If, for instance, they are both thoroughly wetted and then compressed by hand the cotton, representing the compressible soil, remains in the compressed state after the removal of the compressing force, because of capillary tension acting on the surface of the cotton—the same force which stabilized the beach sand referred to above. The capillary tension in this case equals at least the pressure exerted by the hand to compress the cotton and also any tendency possessed by the cotton strands to separate, otherwise the cotton would have expanded. The sponge, representing the elastic subgrades, expands to almost its wet volume upon the removal of the compressing force. This occurs because the tendency of the sponge fabric to expand greatly exceeds the capillary force acting upon the sponge surface. Soaking the sponge prior to compression in a glue whose molecular cohesion exceeds that of water in sufficient amount serves to prevent the elastic rebound upon the removal of pressure.

FIGURE 15.—DETRIMENTAL EFFECTS OF ELASTICITY OF SUBGRADE SOIL. A.—TYPE OF CRACKING LIKELY TO OCCUR IN PAVEMENTS LAID ON IMPROPERLY PREPARED ELASTIC SUBGRADES. NOTE CRACKS REFLECTING RUTS IN SUBGRADE. B.—ALLIGATOR HIDE CRACKING IN MACADAM ROAD SURFACE

Either a macadam or a concrete surface may be seriously damaged by attempts to consolidate elastic subgrades before pavement construction. After the thorough rolling which benefits the compressible subgrades, a subgrade of the elastic type, if it possesses cohesion, is likely to retain a certain degree of compaction. A slight wetting under these conditions, such as is furnished by freshly deposited concrete, may cause a nonuniform rebound of the subgrade. This, combined with water loss from the concrete due to absorption by the soil is likely to cause pavements to crack excessively (fig. 15, A) during the setting period of the concrete.

Movements of heavy material trucks and mixing apparatus adjacent to pavements laid on elastic co-

[3] Permeability is defined as the rate at which gravitational water is transmitted by soils. It depends upon both the hydraulic gradient and the size and number of the soil pores. It varies as the square of the effective diameter of the soil grains. It is expressed as the coefficient of permeability which is designated as k and equals the velocity in centimeters per second under a hydraulic gradient of 1.

hesionless subgrades may cause distortions of the soil supporting the freshly laid concrete sufficient to produce pavement cracking. Cracking of this character may remain in microscopic form for an appreciable period of time. Except during the setting period of the concrete, elastic subgrades are not likely to be detrimental to concrete pavements.

The presence of elasticity in subgrades may prevent macadam pavements from acquiring adequate bond during construction and from retaining it subsequently. Under these conditions macadams may develop "alligator hide" (fig. 15, B) cracking, through which water may pass and cause the subgrade soil to soften and to penetrate the voids of the macadam, thus causing the surface to fail.

FIGURE 16.—FILL BEING CONSTRUCTED OF MATERIAL CONSISTING OF CLODS

DEGREE AND RATE OF COMPRESSION DISCUSSED

The manner in which soils may compress depends to a large extent upon their moisture contents. Those soils whose voids contain air may compress because of either a compression of the entrapped air or the escape of the air from the soil pores. In this case the rate of consolidation depends upon such factors as the resistance of clods (fig. 16) to crushing and can not be computed. The force required to break clods differing in degree of dryness can, however, be investigated in the laboratory.

Soils in the plastic state (fig. 17) or those whose voids are filled with moisture, may consolidate vertically without flowing laterally only when water escapes from the soil pores. Thus foundations supporting buildings and other structures adequately for years settle suddenly when new excavations permit water to escape from the loaded soil supporting the foundations.

In this case the speed of soil consolidation for equal external pressures applied depends primarily upon the permeability of the soil mass. In fact within certain limits it varies directly with the coefficient of permeability of the soil. Since the coefficients of permeability of the soil constituents may vary through a wide range, as shown in Table 3, the rates at which individual soils consolidate may be widely different.

Figure 18 shows how the data furnished by the Terzaghi compression test may be employed to indicate both the amount and the rate of fill settlement. This test has been discussed previously in PUBLIC ROADS by Dr. Charles Terzaghi (*1*).

Water pressed from the soil pat (fig. 18, A) by the weighted piston, passes through the porous stones above and below the pat and escapes from the overflow orifices *a* and *b*. The relation between the voids ratio of the soil and the compressing force is expressed as the load-compression curve, Figure 18, B. The data for constructing the load-compression curve are obtained by applying the compressing force in magnitudes equal approximately to 0, 0.5, 1.5, and 3.0 kilograms per square centimeter and observing the voids ratio produced by each load when applied, until further increase in the deformation of the soil ceases. Consequently the load-compression curve discloses the minimum voids ratio or the maximum density of the soil likely to be produced by loads of given magnitude. Data for constructing the expansion curve (fig. 18, B) are obtained when the load is changed successively from approximately 3.0 to 1.5, 0.5, and 0.0 kilograms per square centimeter and water is allowed to enter the sample. The data for constructing the time-compression curve are furnished by observing the times corresponding to the deformations produced by the individual load increments.

TABLE 3.—*Coefficients of permeability of soil constituents under pressure of 1.5 kilograms per square centimeter*

Soil	Coefficient of permeability
	Centimeter per second
Potomac River sand, 20–100 mesh	18.96×10^{-4}
Mica, 20–100 mesh	0.128×10^{-4}
Rock Creek silt	0.00096×10^{-4}
Diatoms	0.048×10^{-4}
Clay	0.00011×10^{-4}
Peat (Minn.)	0.785×10^{-4}

FIGURE 17.—SETTLEMENT OF ROAD AT BRIDGE APPROACH DUE PRIMARILY TO THE CONSOLIDATION OF THE THOROUGHLY SATURATED UNDERSOIL

The time-compression curve (fig. 18, C) shows the relation between (*a*) the degree of compression of the soil, expressed as a percentage of the total compression occurring in a very long loaded interval, and (*b*) the length of the loaded interval expressed in minutes, the magnitude of the load remaining constant.

In order to demonstrate, by analogy, the method of estimating both the magnitude and the rate of compression caused by a hydraulic fill constructed on inundated river bottom land as illustrated in Figure 18, D, it must be assumed first that the compression is direct and no lateral thrust is involved; second, that both the fill material on top and the hard compact sand beneath are more permeable than the soft soil layer; third, that the moisture is free to pass through the underlayer of

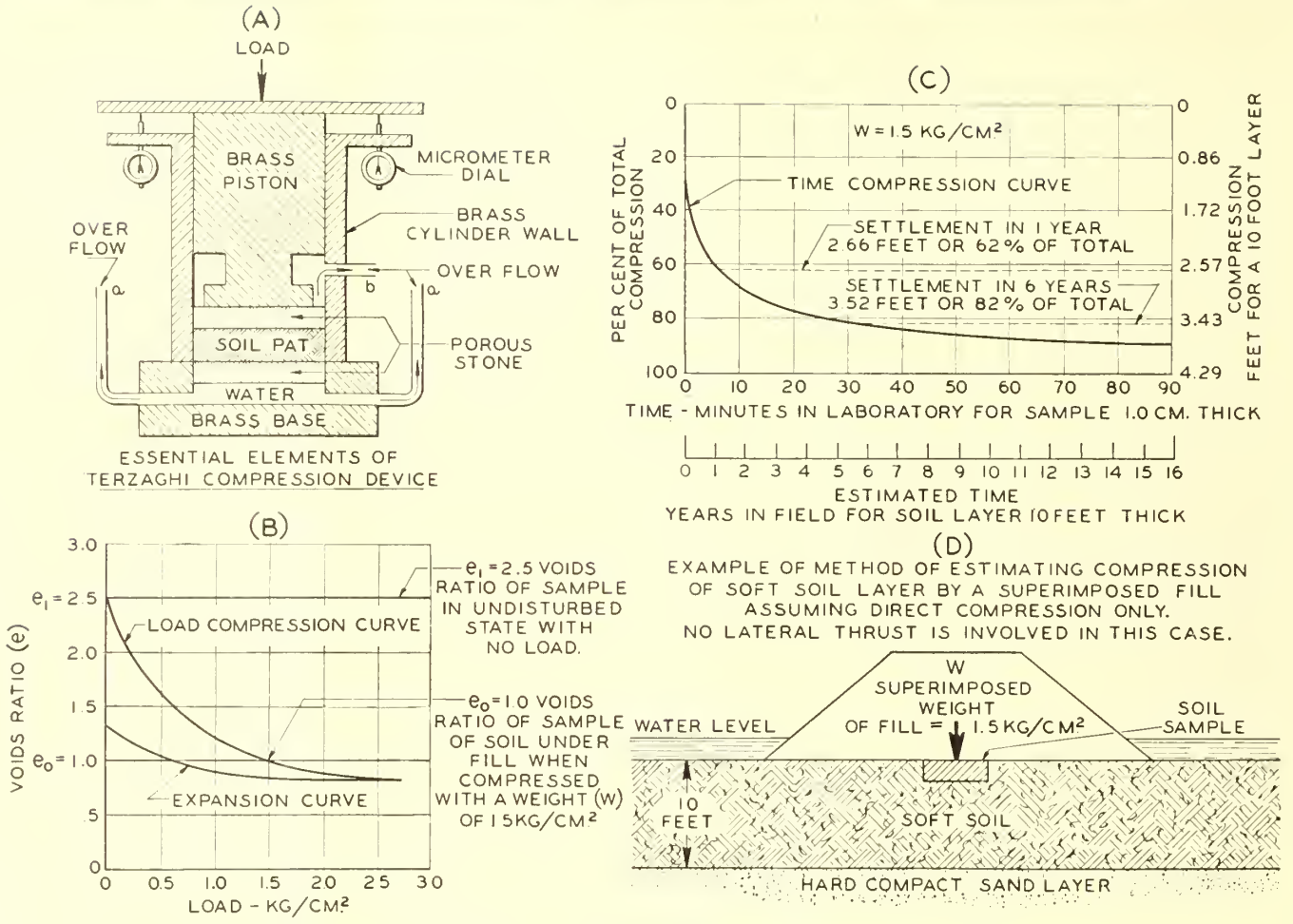

(A)
LOAD
BRASS PISTON
MICROMETER DIAL
BRASS CYLINDER WALL
OVER FLOW
OVER FLOW
SOIL PAT
POROUS STONE
WATER
BRASS BASE
ESSENTIAL ELEMENTS OF TERZAGHI COMPRESSION DEVICE
(B)
VOIDS RATIO (e)
e_1 = 2.5
e_0 = 1.0
LOAD COMPRESSION CURVE
EXPANSION CURVE
e_1 = 2.5 VOIDS RATIO OF SAMPLE IN UNDISTURBED STATE WITH NO LOAD.
e_0 = 1.0 VOIDS RATIO OF SAMPLE OF SOIL UNDER FILL WHEN COMPRESSED WITH A WEIGHT (W) OF 1.5 KG/CM²
LOAD - KG/CM²
(C)
W = 1.5 KG/CM²
TIME COMPRESSION CURVE
SETTLEMENT IN 1 YEAR 2.66 FEET OR 62% OF TOTAL
SETTLEMENT IN 6 YEARS 3.52 FEET OR 82% OF TOTAL
PER CENT OF TOTAL COMPRESSION
COMPRESSION FEET FOR A 10 FOOT LAYER
TIME - MINUTES IN LABORATORY FOR SAMPLE 1.0 CM. THICK
ESTIMATED TIME YEARS IN FIELD FOR SOIL LAYER 10 FEET THICK
(D)
EXAMPLE OF METHOD OF ESTIMATING COMPRESSION OF SOFT SOIL LAYER BY A SUPERIMPOSED FILL ASSUMING DIRECT COMPRESSION ONLY. NO LATERAL THRUST IS INVOLVED IN THIS CASE.
W SUPERIMPOSED WEIGHT OF FILL = 1.5 KG/CM²
WATER LEVEL
SOIL SAMPLE
10 FEET
SOFT SOIL
HARD COMPACT SAND LAYER

FIGURE 18.—DATA FURNISHED BY THE TERZAGHI COMPRESSION TEST

sand, and last, that the moisture content throughout the thickness of the soft soil layer is uniform.

Under these conditions the soft soil layer in the river bottom (fig. 18, D) is subjected to compression similar to that acting on the soil pat (fig. 18, A). Thus the weight of the fill material corresponds to the compressing force, and the fill material on top and the compact sand beneath the soft soil layer correspond to the porous stones. (Fig. 18, A). In addition let the load-compression curve (fig. 18, B) and the time-compression curve (fig. 18, C) be assumed to represent tests performed upon an undisturbed sample of the soft river bottom soil, as indicated in Figure 18, D.

Under the weight of the fill material (1½ kilograms per square centimeter), the undersoil will, since the assumed conditions are the same as those existing in the laboratory, compress from a voids ratio $e_1 = 2.5$ (original undisturbed state) to a voids ratio $e_0 = 1$. The ratio of soil thickness after compression to soil thickness before compression equals the ratio of soil volume (soil particles plus voids) after compression to the soil volume before compression, or $\frac{1+e_0}{1+e_1}$. Consequently the soil layer 10 feet thick will compress to a layer whose thickness is given by the expression

$$\frac{1+e_0}{1+e_1} \times 10 = 5.71 \text{ feet.}$$

The time in years required for different stages of settlement is computed from the time-compression curve on the assumption that the time required for two soil layers to compress in equal degree varies as the squares of the thickness of the layers.

Accordingly Figure 18 informs us (*a*) that the fill will settle 4.29 feet and (*b*) that 2.66 feet of this amount will occur during the first year and that an additional settlement of 0.86 foot will occur during the succeeding five years.

CAPILLARITY THE IMPORTANT AGENT CAUSING CHANGES OF WEATHER TO BE REFLECTED IN SUBGRADE MOVEMENTS

The more important subgrade movements due to climatic influences are (*a*) expansion of the soil occurring with an increase in moisture content (*b*) shrinkage of the soil occurring with a decrease in moisture content, and (*c*) heaving of the soil during frost.

These occurrences depend upon the physical phenomenon capillarity, which, so far as subgrades are concerned, is defined as the ability of soils to transmit moisture in a finely divided state in all directions in spite of both the direction in which gravity acts and the force of gravity.

Capillary action is illustrated in Figure 19. Capillarity draws the liquid up through the cheesecloth wicks, over the edge of the container A, and down to the ends of the wicks on the outside of the container. This merely moistens the wick on the left whose outside end is just about on a level with the surface of the liquid in the container. In addition to becoming moist, the wick on the right, the outside end of which is located appreciably

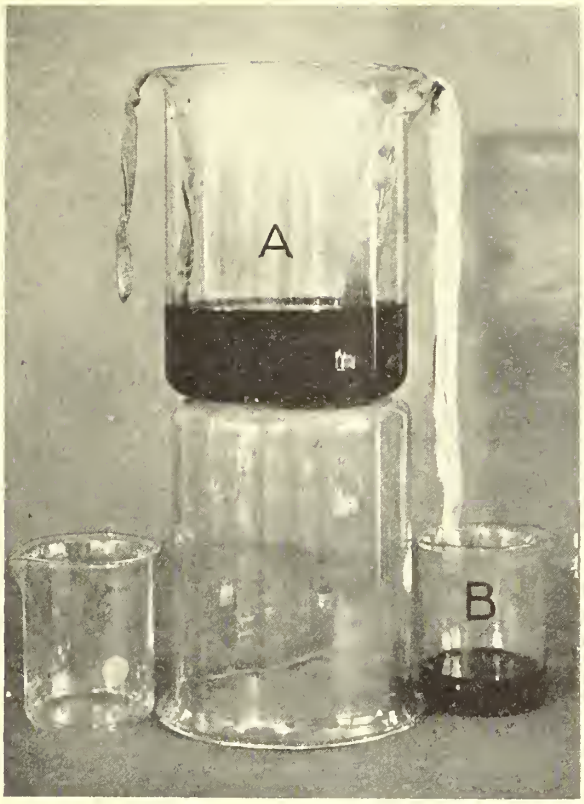
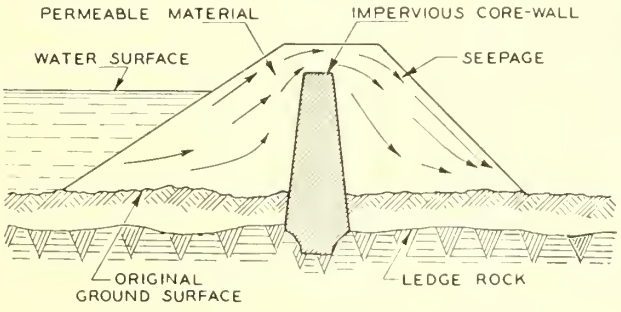


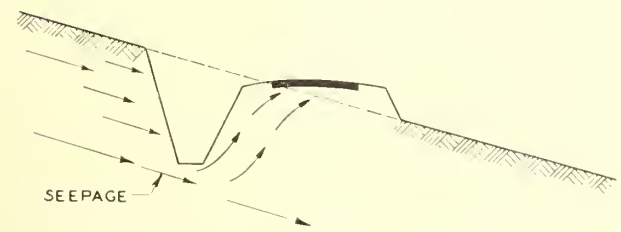
FIGURE 19.—ILLUSTRATION OF CAPILLARITY

below the surface of the liquid in the container, performs like a syphon and transfers the liquid from container A to container B.

When this photograph was taken the liquid was dropping into container B from the end of the cheesecloth wick at the rate of 1 drop every 38 seconds or



A - EARTH DAM WITH CORE-WALL



B - HIGHWAY ON SIDE-HILL LOCATION

FIGURE 20.—ILLUSTRATIONS OF CAPILLARY ACTION IN SOILS

1 gram every 10 minutes. This equals 1 gallon every 631 hours. At times during the experiment the rate was as much as 1 gallon every 12 days.

This simple experiment explains the occurrence of seepage in the lower face of an earth dam, even though the top of the impervious corewall is higher than the adjacent water surface, Figure 20, A, and in subgrades in spite of ditches, as shown in Figure 20, B.

The maximum distance through which water may be forced by capillarity depends upon the surface tension of the water and the size of the soil pores, and increases as the size of the soil pores decreases, the temperature of the water remaining constant (*1*, *5*). The rate at which capillary moisture travels depends upon the capillary tension, upon the frictional resistance furnished by the walls of the pores to the flow of water, and upon the rate at which capillary equilibrium is destroyed by evaporation, the formation of ice crystals, or change in ground water elevation (*11*).

FIGURE 21.—SOIL IN FACE OF CUT ERODED BECAUSE OF ITS EXPANSIVE PROPERTIES

PROPERTIES AFFECTING EXPANSION AND SHRINKAGE OF SOILS ANALYZED

The extent to which water will be absorbed depends upon both the capillary properties and the degree of cohesion possessed by the soil. Water entering cohesionless soils through capillary action may cause the grains to separate to such an extent that the soil quickly disintegrates, as shown in Figure 21. A sufficient amount of cohesion existing between the soil particles will prevent the entrance of water in an amount sufficient to cause the soil to lose stability, unless the soil is manipulated. The amount of cohesion possessed by a soil of given constituents depends upon both the moisture content and the state of compaction of the soil. Therefore the relative amount to which the soil will expand depends upon both the degree of consolidation and the moisture content of the soil before wetting.

Each of 90 soil cakes were compressed in the wet state in the subgrade laboratory of the Bureau of

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Public Roads, under a load of 3 kilograms per square centimeter. The load was then reduced to 0.028 kilogram per square centimeter and the cakes were permitted to absorb water. Subsequently two disks, each 1 square inch in area, were cut from each of the 90 soil cakes. One of these disks, in the wet state, was immersed in water and its counterpart was first allowed to dry to constant weight in the air and was then immersed in water.

Eight of the 90 disks, containing less than 12 per cent clay and immersed in the wet state disintegrated after being immersed for periods averaging 73 days. Of the 8 corresponding disks immersed in the dry state, 6 disintegrated after being immersed for an average period of 7 minutes and the remaining 2 swelled and cracked in appreciable amount but did not completely disintegrate after being immersed for a period of 25 days. Sixty-eight of the 90 disks containing, with several exceptions, clay 13 to 77 per cent and immersed in the wet state, remained intact after being immersed for an average period of nine months. Of the corresponding 68 disks immersed in the dry state, 26 disintegrated after being immersed for an average period of 10 minutes, 41 disintegrated after being immersed for an average period of 1 hour, and the remaining disk cracked and swelled in appreciable amount after being immersed for a period of 10 days.

As additional evidence that cohesion in soils tends to prevent their expansion and disintegration due to water absorption reference is made to Figure 22. The soil cakes shown in this figure were made up from soil

FIGURE 22.—SOIL SAMPLES CONTAINING WATER-GAS TAR IN VARYING AMOUNTS, PHOTOGRAPHED AFTER IMMERSION IN WATER FOR A PERIOD OF TWO WEEKS

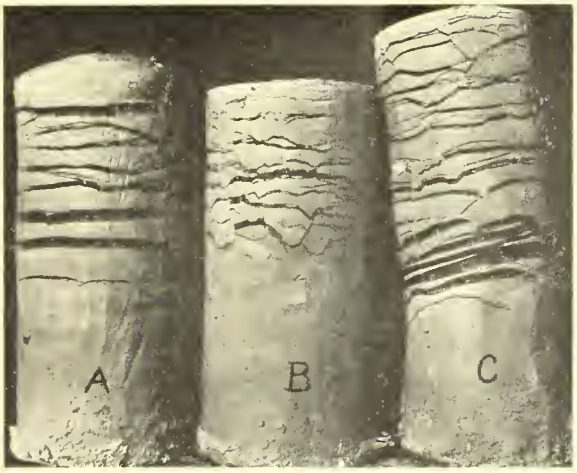
samples taken at different depths in a subgrade located at Arlington, Va., which was treated with water-gas tar in 1923. After their removal from the subgrade in 1929, these cakes were compressed in a semidry state, dried to constant weight in the air, and immersed in water for a period of two weeks. The two cakes shown on the left of Figure 22 contain tar in appreciable amount and exhibit no signs of disintegration. The third cake from the left contains but a small amount of tar and has crumbled slightly along the top edges. The fourth cake from the left contains but a slight trace of tar and shows crumbling in appreciable amount near the top. The two cakes on the right contain no visible trace of tar and have crumbled to a still greater extent.

Reduction in soil volume as illustrated in Figure 23 is caused by reduction in the moisture content of the soil. This action depends upon the capillary force exerted as the moisture content is reduced by evaporation and upon the resistance furnished by the soil particles to being consolidated. The theory of shrinkage in soils may be briefly stated as follows (*12*): Assume a soil with its pores completely filled with moisture to be in the liquid state. Under these conditions the contractive force exerted by the surface tension of the water is practically zero. As water evaporates from

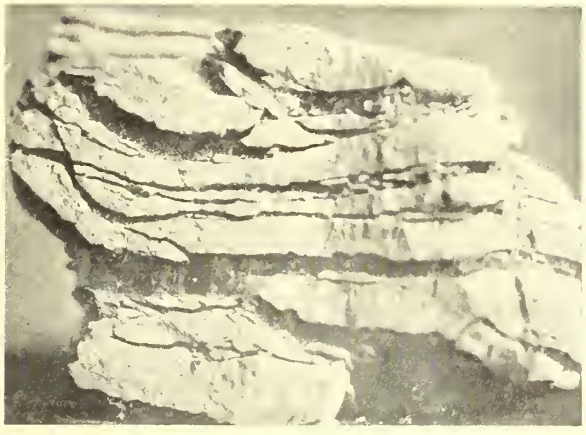
FIGURE 23.—LARGE SHRINKAGE CRACKS OVER 3 INCHES WIDE AND OVER 1 FOOT DEEP

the sample the capillary tension exerts on the outer of surface of the sample a uniformly distributed force acting at every point perpendicular to the outer surface of the sample, and tending to draw the particles in. As the soil sample becomes smaller and smaller its resistance to further shrinkage correspondingly in-

FIGURE 24.—EXAMPLES OF FROST DAMAGE IN ROADS. A.—GRAVEL ROAD HEAVED BY FROST ACTION. B.—BITUMINOUS MACADAM ROAD DAMAGED BY FROST ACTION IN SUBGRADE



Ice lenses in test cylinders frozen by Prof. Stephen Taber



Ice lenses observed by F. C. Lang in soil in Minnesota

FIGURE 25.—EXAMPLES OF ICE SEGREGATION IN SOILS

creases. Finally the soil attains a volume at which the resistance of the soil sample to further reduction in volume just equals the capillary pressure exerted by the evaporating moisture. Further evaporation will not appreciably decrease the volume of the soil sample. The moisture content of the soil at this state of equilibrium is termed "the shrinkage limit" and is discussed in Part II of this report, to be published in the next issue of PUBLIC ROADS.

DETRIMENTAL HEAVING CAUSED BY SEGREGATION OF WATER WHEN FREEZING

Heaving of soils due to frost action (fig. 24, A and B) is caused by an increase in total moisture content occurring as ice layers or crystals (*13*, *14*, *15*). Its magnitude depends upon the rate of temperature change, the moisture content of the soil prior to freezing, the proximity of additional water and the rate at which capillary flow occurs (*13*, *14*, *15*, *16*).

The formation of well defined ice layers in the soil (figs. 25,[4] and 26) depends on three physical phenomena—

(*a*) The tendency of water particles contained in soil pores of the larger capillary dimensions to freeze at either normal freezing or slightly less than normal freezing temperatures (−1°, −4° C.).

(*b*) The tendency of water particles contained in soil pores of the smaller capillary dimensions to resist freezing at abnormally low temperatures (as low as −70° C.).

(*c*) The tendency of water particles of freezable size, during the process of freezing, to draw to themselves from adjacent fine capillaries the small particles of water which individually do not freeze at ordinary freezing temperatures (*14*). When drawn to the existing ice crystal these small water particles freeze and increase in size. Continuation of this process causes

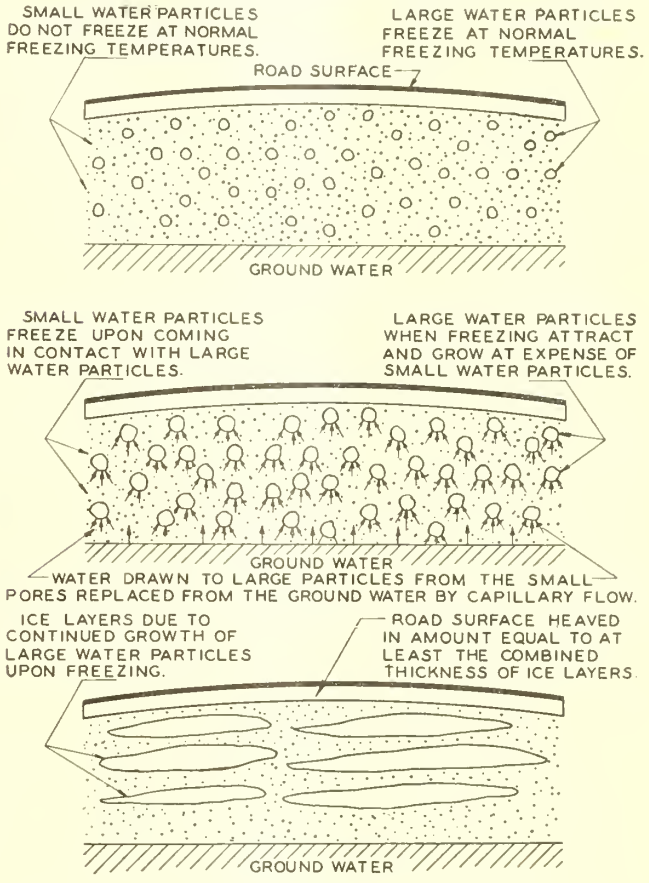

SMALL WATER PARTICLES DO NOT FREEZE AT NORMAL FREEZING TEMPERATURES.
LARGE WATER PARTICLES FREEZE AT NORMAL FREEZING TEMPERATURES.
ROAD SURFACE
GROUND WATER
SMALL WATER PARTICLES FREEZE UPON COMING IN CONTACT WITH LARGE WATER PARTICLES.
LARGE WATER PARTICLES WHEN FREEZING ATTRACT AND GROW AT EXPENSE OF SMALL WATER PARTICLES.
GROUND WATER
WATER DRAWN TO LARGE PARTICLES FROM THE SMALL PORES REPLACED FROM THE GROUND WATER BY CAPILLARY FLOW.
ICE LAYERS DUE TO CONTINUED GROWTH OF LARGE WATER PARTICLES UPON FREEZING.
ROAD SURFACE HEAVED IN AMOUNT EQUAL TO AT LEAST THE COMBINED THICKNESS OF ICE LAYERS
GROUND WATER

FIGURE 26.—DIAGRAM ILLUSTRATING THE PHYSICS OF FROST HEAVE

the original ice crystals to increase in size as long as they are being supplied with small water particles drawn up through the fine capillaries from the ground water supply.

Whether or not frost heave will occur depends upon the quantity of moisture capable of being raised to a given height above the water table in a given time. Neither the height to which water will rise by capillarity nor the rate of such rise is alone the determining factor. Just as the amount of water furnished by a pipe depends upon the pressure acting on the water, the diameter of the pipe and the frictional resistance to flow, the raising of a quantity of water sufficient to produce frost heave in the subgrade at a given height above the ground water elevation (disregarding the rate at which capillary equilibrium is destroyed) depends upon the force of capillarity, the area of pore space, and the frictional resistance.

Capillary pressure varies inversely with the diameter of the pores. The frictional resistance to flow through a soil is a function of the surface area of the soil particles and consequently increases with a decrease in grain size at a much greater rate than does the capillary pressure. Therefore, in order to furnish capillary

[4] Reproduced from the article, Freezing and Thawing of Soils as Factors in the Destruction of Road Pavements, by Stephen Taber, Public Roads, vol. 11, No. 6, August, 1930.

moisture in detrimental amounts the pore size must be small enough to furnish appreciable capillary pressure but large enough to prevent too much frictional resistance to flow.[5]

As an indication of the amounts of water furnished by capillarity, it has been estimated that certain Iowa silts (*9*) are capable of raising water by capillarity at the rate of about 5 feet per year. It is indicated by observations in both New Hampshire and Minnesota that in extreme cases the rate may be as high as 10 or 15 feet per year.

FIGURE 27.—FROZEN CYLINDER, HALF SAND AND HALF CLAY. MUCH SEGREGATED ICE IN CLAY BUT NOT IN SAND. FURNISHED BY TABER EXPERIMENTS

Generally it can be stated that in cohesionless soils possessing but little capillarity no important frost heaves occurs because practically all of the contained water freezes at normal freezing temperatures and small unfrozen water particles do not exist in amounts sufficient to cause the frozen particles to suffer appreciable growth. Soils possessing relatively high capillarity and relatively low cohesion are likely to heave in very appreciable amounts. The contrast between sand and clay in this respect is shown in Figure 27.[4]

Highly cohesive soils may possess very high capillarity but the resistance to water flow in these soils is very great. Consequently in dense cohesive soils with low ground water level and absence of lateral seepage, only limited amounts of water are available for ice segregation. Under these conditions the soil adjacent to the growing ice crystals, as illustrated in Figure 28, is likely to dry out and shrink because of the loss of moisture (*16*). The ground water elevation in clays, therefore, must be comparatively high in order that important frost heave may occur.

IMPORTANT SUBGRADE CHARACTERISTICS INDICATED BY THE PRESENCE OF CERTAIN SOIL CONSTITUENTS

In the foregoing discussion certain relations have been shown to exist between the five basic physical characteristics of a soil (internal friction, cohesion, compressibility, elasticity, and capillarity) and the important characteristics of subgrade performance, i. e., resistance to lateral flow, the property of compressing vertically under applied loads with or without rebound upon the removal of load, resistance to sliding in cuts and fills, shrinkage or expansion due to changes in moisture, and heaving under frost action.

Both the state in which the soil exists and the properties of the soil constituents exert an important influence upon the occurrence of these characteristics. Thus, for instance, tests performed by Terzaghi (*4*) disclose that the ultimate bearing value (yield point) of a soil in the undisturbed state may be over twice that of the same soil in the disturbed state. Likewise the compressibility may be different when soil is in the disturbed state from what it is in the undisturbed state (*17*).

It is well known that the faces of cuts in certain of the loess soils in the Middle West may stand vertically for years in the undisturbed state whereas these same soils in the disturbed state lose stability easily and flow in the presence of water.

It is also common knowledge that sands may be either highly stable or "quick" depending on whether water, as for instance that furnished by waves, flows downward or, as in the case of that furnished by springs, flows upward through them.

In spite of these facts the importance of tests performed on soils in the disturbed state becomes apparent when one considers (*a*) that the soil composing the subgrade generally, exists in at least a partially manipulated state and that composing the sands-clay and other low type road surfaces exists in a completely manipulated state, and (*b*) that the constants furnished by such tests serve to disclose the presence of those soils constituents which exert an important influence not only

FIGURE 28.—TYPE OF SHRINKAGE CRACKING LIKELY TO OCCUR WHEN WATER IS DRAWN OUT OF SOILS DUE TO FREEZING IN ADJACENT AREAS

on the five basic physical characteristics of subgrades referred to above but also upon the state in which soils may occur in the soil profile. When only the mechanical analysis was used to identify soils, many soils indicated by this determination to be similar in character when disturbed, were observed to be radically different in character when undisturbed in the field. This difference in field behavior was attributed to difference in the environment under which the soil developed or existed. Now it is found that soils similar in character according to the mechanical analysis may differ widely in character according to the physical tests and furthermore that many differences in field behavior of soils are due to differences in soil constituents.

The subgrade investigations have made it increasingly evident that the states in which soils are likely to exist under specified conditions, as well as the physical characteristics of the soils in different states, may be largely dependent upon the presence of certain soil constituents.

Among the many natural constituents of soils, a comparatively few serve to illustrate the properties which

[4] Reproduced from the article, Freezing and Thawing of Soils as Factors in the Destruction of Road Pavements, by Stephen Tabor, Public Roads, vol. 11, No. 6, August, 1930.

[5] In 1909 Atterberg determined experimentally that the maximum height of capillary rise in 24 hours occurs in soil consisting primarily of particles 0.02 millimeter in diameter. This result was confirmed theoretically by Terzaghi (Erdbaumechanik, 1926, fig. 23).

may exert an important influence upon subgrade performance. These representative soil constituents are:

Gravel.—Particles larger than 2.0 millimeters in diameter (No. 10 sieve).

Coarse sand.—Particles between 0.25 (No. 60 sieve) and 2.0 millimeters in diameter.

Fine sand.—Particles between 0.05 and 0.25 millimeter in diameter.

Silt.—Particles between 0.005 and 0.05 millimeter in diameter.

Cohesive clay.—Particles smaller than 0.005 millimeter in diameter.

Gluey colloids.—According to Albert Atterberg (*18*), soil particles 0.002 millimeter or smaller in diameter show pronounced Brownian movements when suspended in water. This phenomenon, as explained by Oscar Edward Meinzer, indicates that the colloidal stage of fineness is reached at this size and, according to the kinetic theory of heat, the particles are so small that they are bounced about by the rapidly moving molecules with which they collide. The colloidal fraction reported in the mechanical analysis, however, consists of particles 0.001 millimeter and smaller in size.

Mica flakes.

Diatoms.

Peat.

Chemical constituents.—Certain chemicals, such as lime and magnesium, have a tendency to flocculate [6] fine-grained soils; others, such as sodium and potassium have a tendency to deflocculate or disperse fine-grained soils. (Fig. 29.)

Of these constituents, certain of which are shown in the photomicrographs of Figure 30, the gravel and sand are indicative of high internal friction, the silt, peat, and diatoms of detrimental capillarity; the clay and colloidal glues are indicative of cohesion and, together with the silt and when not flocculated, of compressibility; and the mica flakes, peat, and flocculated soils are indicative of elasticity.

Generally gravel and coarse sand furnish the main hardness and supporting strength of graded soil roads, especially in wet weather. Fine sand adds an embedment support to the coarse sand, and silt with low moisture content adds embedment for the sand. Clay and colloidal glues furnish cohesive and adhesive bond variable with their moisture contents (*11*).

The curves of Figure 31 show the great range of compressibility and expansion possessed by different soil constituents.

SUBGRADES TENTATIVELY ARRANGED IN GROUPS

Because of the fact that the presence of certain soil constituents indicates the important soil properties, the subgrades may be arranged in groups representative of both soil constituents and characteristics. On this basis, the various soils have been tentatively arranged in groups with respect to their performance when used as subgrades, as follows (*7*):

UNIFORM SUBGRADES

Group A–1.—Well-graded material, coarse and fine, excellent binder. Highly stable under wheel loads, irrespective of moisture conditions. Functions satisfactorily when surface treated or when used as a base for relatively thin wearing courses.

Group A–2.—Coarse and fine materials, improper grading or inferior binder. Highly stable when fairly dry. Likely to soften at high water content caused either by rains or by capillary rise from saturated lower strata when an impervious cover prevents evaporation from the top layer, or to become loose and dusty in long-continued dry weather.

Group A–3.—Coarse material only, no binder. Lacks stability under wheel loads but is unaffected by moisture conditions. Not likely to heave because of frost nor to shrink or expand in appreciable amount. Furnishes excellent support for flexible pavements of moderate thickness and for relatively thin rigid pavements.

Group A–4.—Silt soils without coarse material, and with no appreciable amount of sticky colloidal clay. Has a tendency to absorb water very readily in quantities sufficient to cause rapid loss of stability even when not manipulated. When dry or damp, presents a firm riding surface which rebounds but very little upon the removal of load. Likely to cause cracking in rigid pavements as a result of frost heaving, and failure in flexible pavements because of low supporting value.

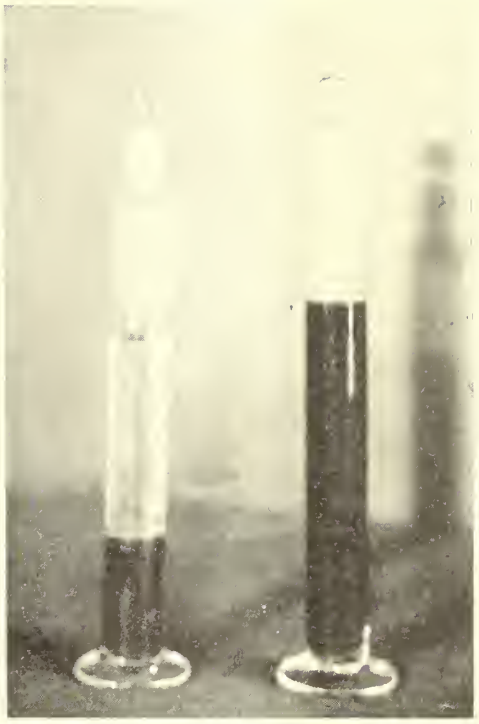


FIGURE 29.—SOIL IN FLOCCULATED AND DISPERSED STATE AFTER 24 HOURS SEDIMENTATION. FLOCCULATED SOIL (LEFT) SETTLES OUT RAPIDLY LEAVING CLEAR LIQUID ABOVE. DISPERSED COLLOIDS REMAIN IN SUSPENSION CAUSING LIQUID TO BE CLOUDY

Group A–5.—Similar to Group A–4, but furnishes highly elastic supporting surfaces with appreciable rebound upon removal of load even when dry. Elastic properties interfere with proper compaction of macadams during construction and with retention of good bond afterwards.

Group A–6.—Clay soils without coarse material. In stiff or soft plastic state absorb additional water only if manipulated. May then change to liquid state and work up into the interstices of macadams or cause failure due to sliding in high fills. Furnish firm support essential in properly compacting macadams only at stiff consistency. Deformations occur slowly and removal of load causes very little rebound. Shrinkage properties combined with alternate wetting and drying under field conditions are likely to cause cracking in rigid pavements.

Group A–7.—Similar to Group A–6, but at certain moisture contents deforms quickly under load and rebounds appreciably upon removal of load, as do subgrades of Group A–5. Alternate wetting and drying under field conditions leads to even more detrimental volume changes than in Group A–6 subgrades. May cause concrete pavements to crack before setting and to crack and fault afterwards. May contain lime or associated chemicals productive of flocculation in soils.

Group A–8.—Very soft peat and muck incapable of supporting a road surface without being previously compacted.

NONUNIFORM SUBGRADES

Soils of these groups cause concrete pavements to crack or fault excessively and flexible types to fail or to develop rough riding surfaces.

Group B–1.—Nonuniform natural ground due to abrupt variation in soil characteristics or soil profile, or to frequent change in field conditions.

Group B–2.—Nonuniform subgrade due to nonuniform composition of fill.

Group B–3.—Nonuniform subgrade consisting in part of natural ground and in part of fill materials.

[6] Arrangement of a number of soil colloids into groups having approximately the size of silt particles.

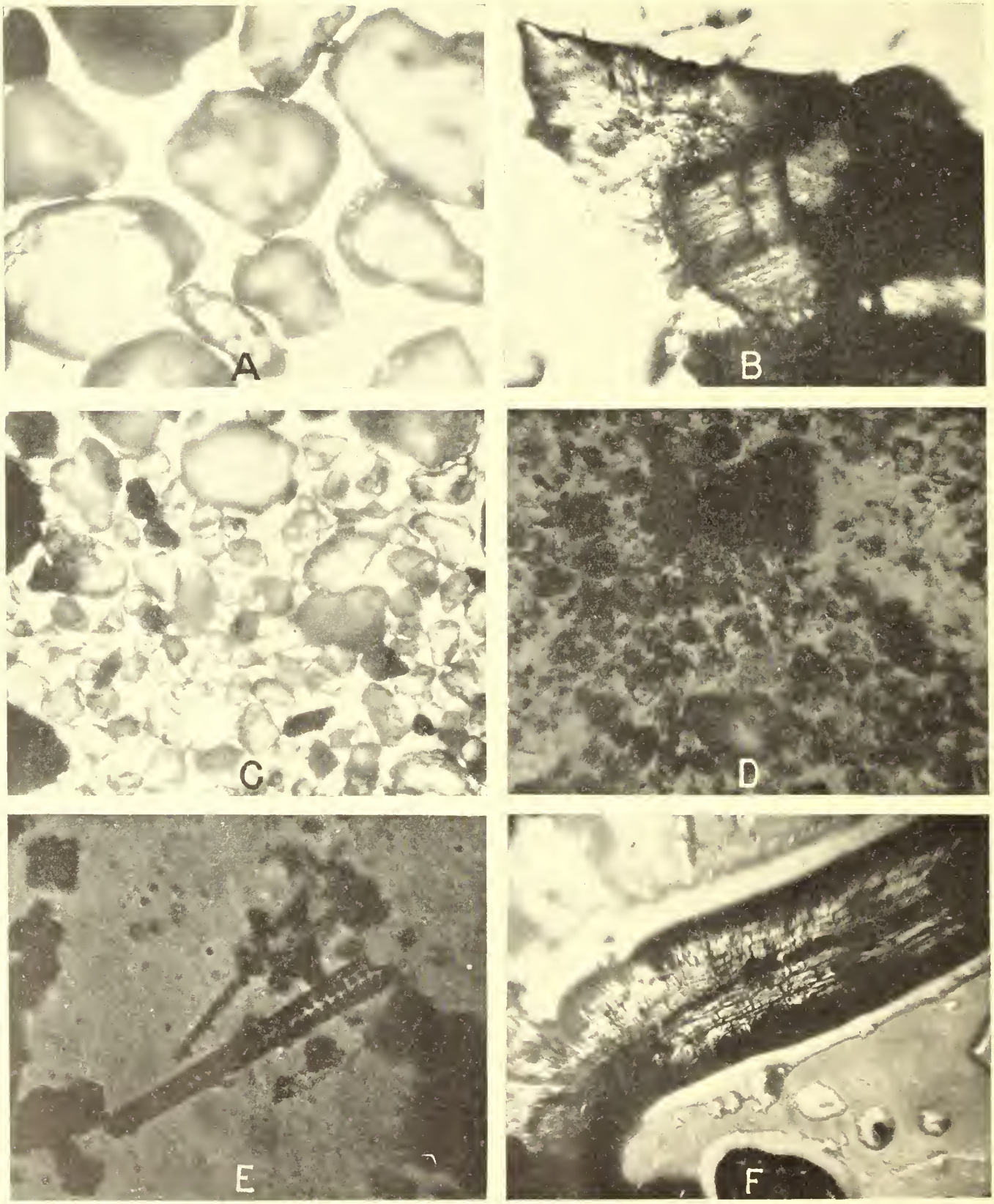


FIGURE 30.—PHOTOMICROGRAPHS OF SOIL CONSTITUENTS. A.—BEACH SAND. B.—ANGULAR SAND GRAIN. C.—GLACIAL SAND. D.—SOIL CONTAINING DIATOMS. NOTE SPONGELIKE APPEARANCE. E.—SINGLE DIATOMS. F.—PEAT-BOG MATERIAL. NOTE FIBROUS STRUCTURE AND FILM OF WATER SURROUNDING INDIVIDUAL PARTICLES

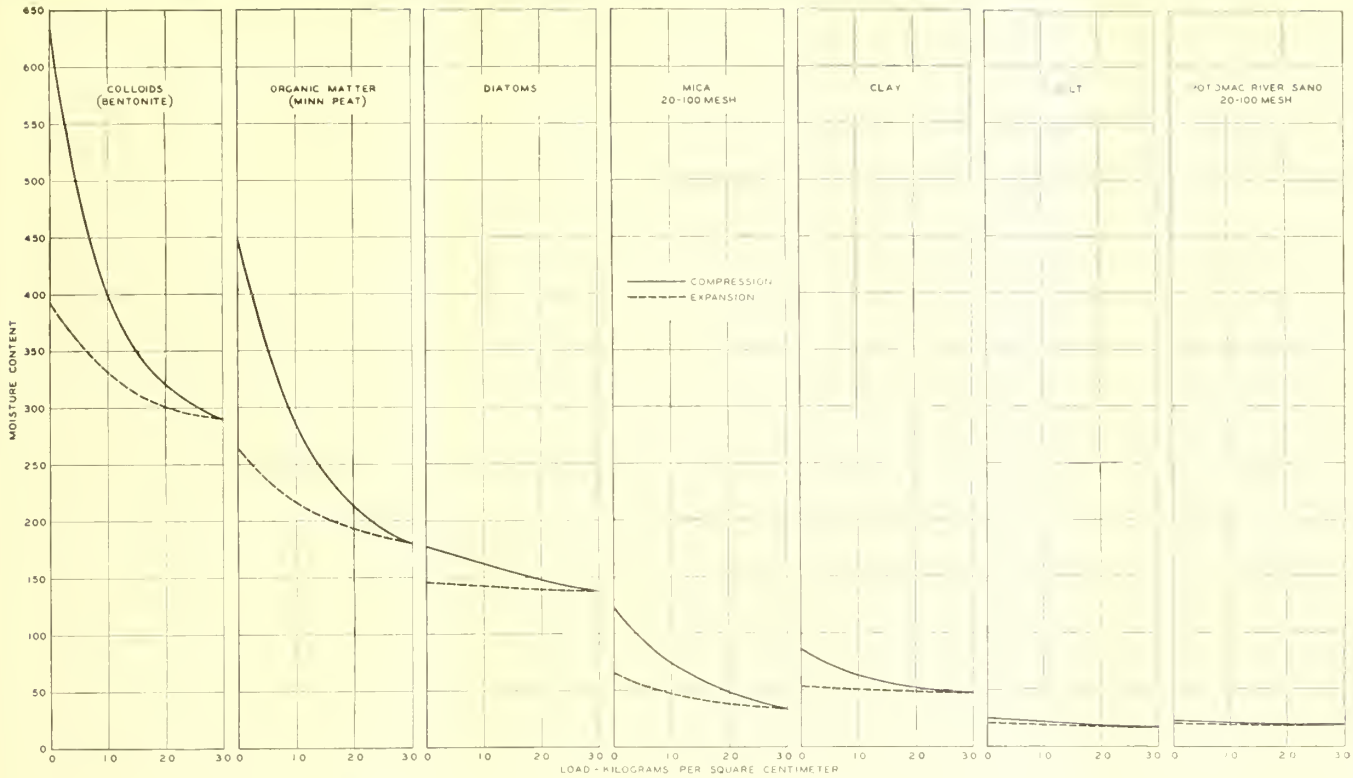


FIGURE 31.—COMPRESSION AND EXPANSION CURVES FOR REPRESENTATIVE SOIL CONSTITUENTS

The foregoing is not intended to be a rigid and final soil classification. It does, however, arrange the uniform soils according to those physical characteristics which are important with respect to subgrade performance and in this manner constitutes the basis for a final classification.

The terms "clay," "silt," and "sand" are used in defining the different groups. These terms, however, refer to the physical properties generally assumed to be possessed by these constituents rather than to definite grain sizes. Thus in the identification of members of the different soil groups grain size is subordinate to physical properties.

The advantage furnished by this method of identification is illustrated as follows: Assume for instance a bulky grained material, Figure 32, A and B, passing the No. 20 and retained on the No. 100 sieve. According to grain size it is sand. Physically it furnishes excellent firm support for a road surface. It does not possess either detrimental elasticity or capillarity and is, consequently, a Group A–3 subgrade.

A similar material containing an appreciable amount of micaceous particles, Figure 32 C, passing the No. 20 and retained on the No. 100 sieve would also be sand, according to grain size. Physically, however, this material possesses both elasticity and capillarity in detrimental amounts; and, since it has no cohesion, it is a Group A–5 subgrade.

In this grouping, therefore, sand instead of being a material characterized only by a specific grain size becomes a Group A–3 subgrade having internal friction, no cohesion, and capillarity in amount insufficient to cause detrimental expansion. Members of the A–3 group are identified with respect to the ability possessed by their grains to resist sliding over each other.

Silt instead of being a material characterized only by a given grain size is divided into two groups: A–4 subgrade, which possesses internal friction, capillarity in appreciable amount, and neither cohesion nor elasticity in appreciable amount; and A–5, subgrade which has internal friction, both capillarity and elasticity in appreciable amount, and no cohesion.

Likewise, the clays are divided into two groups: A–6 subgrade which possesses both cohesion and capillarity in appreciable amount, but neither internal friction nor elasticity; and A–7 subgrade which possesses cohesion, capillarity, and elasticity in appreciable amount, but no internal friction.

This grouping with respect to both the physical properties of the soil and the soil constituents is illustrated diagrammatically in Figure 33. Different positions on the diagram represent different combinations of the five basic physical soil properties. Thus, for instance, a point plotted in the center of the diagram represents the perfect A–3 subgrade, having a maximum amount of internal friction. As the position of the point shifts from the center to the outer boundary of the diagram the magnitude of internal friction decreases gradually from a maximum to negligible amounts. As a point shifts along the circumference from the bottom to the top of the chart the indicated soil changes gradually from the compressible to the elastic type.

Adding compressible materials in increasing amounts therefore gradually changes a group A–3 sand first to a nonplastic variety of the A–2 subgrade; second, to a well graded A–1 subgrade; third, to a plastic variety of the A–2 subgrade and last to either a Group A–4 or Group A–6 subgrade.

The friable variety (sand predominating) differs from the plastic variety (clay predominating) of the Group A–2 subgrade in the following manner. A friable variety, to remain stable, requires the cohesion furnished by capillary pressure and therefore is likely to be highly stable on damp and unstable on thoroughly

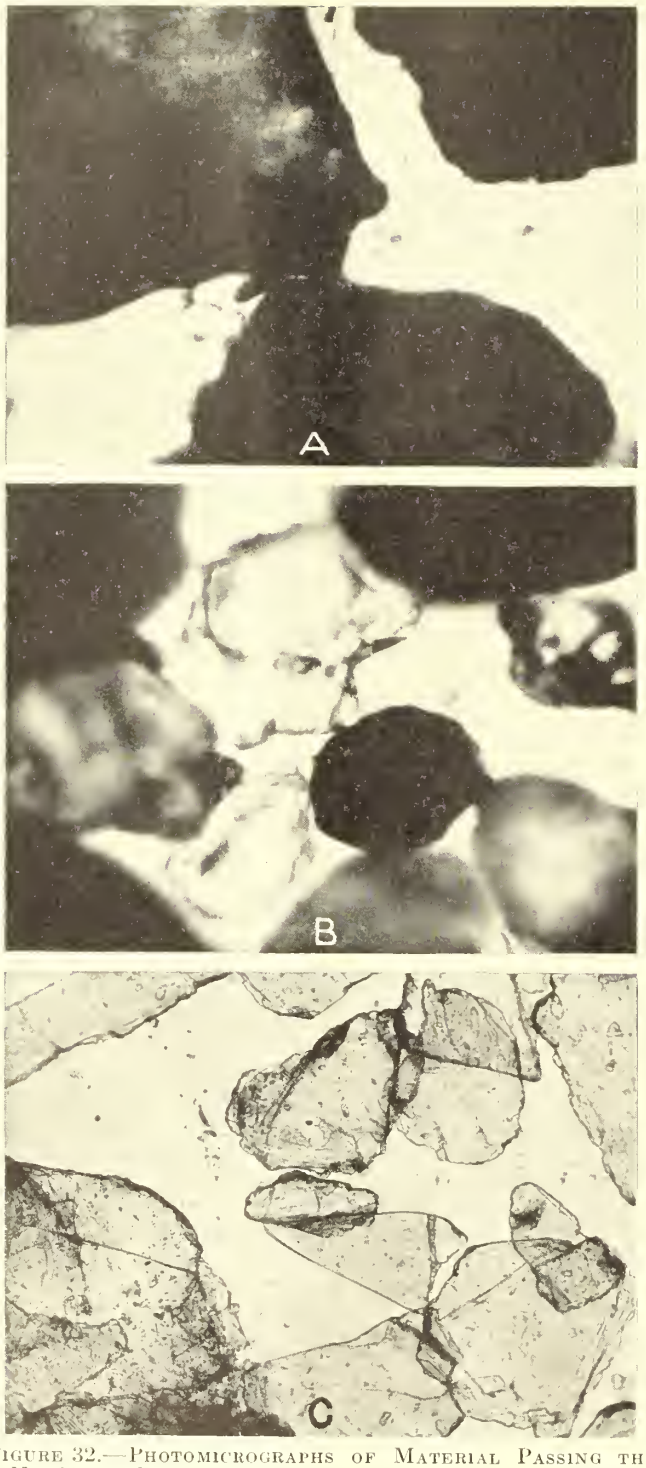


FIGURE 32.—PHOTOMICROGRAPHS OF MATERIAL PASSING THE NO. 20 AND RETAINED ON THE NO. 100 SIEVE. A.—CRUSHED ANGULAR SAND. B.—SUBANGULAR RIVER SAND. C.—MICA FLAKES

dry subgrades. The plastic variety, in contrast, remains stable when fairly dry and is apt to soften on damp subgrades.

Thus one sees that the uniform subgrade groups may be defined with respect to properties summarized as follows:

Group A–1.—High internal friction, high cohesion, no detrimental shrinkage, expansion, capillarity, or elasticity.

Group A–2.—High internal friction and high cohesion only under certain conditions. May have detrimental shrinkage, expansion, capillarity, or elasticity.

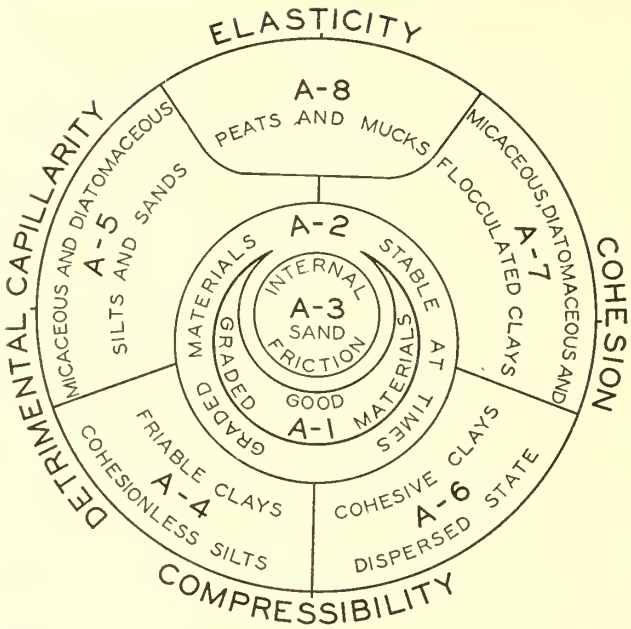


FIGURE 33.—DIAGRAM ILLUSTRATING CHARACTERISTICS OF UNIFORM SUBGRADE GROUPS

Group A–3.—High internal friction, no cohesion, no detrimental capillarity, or elasticity.

Group A–4.—Internal friction variable, no appreciable cohesion, no elasticity, capillarity important.

Group A–5.—Similar to A–4 and in addition possesses elasticity in appreciable amount.

Group A–6.—Low internal friction, cohesion high under low moisture content, no elasticity, likely to expand and shrink in detrimental amount.

Group A–7.—Similar to A–6 but possesses elasticity also.

Group A–8.—Low internal friction, low cohesion, apt to possess capillarity and elasticity in detrimental amount.

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(Continued on page 116)

THE EFFECT OF WATER-GAS TAR ON THE STRENGTH AND ALKALI RESISTANCE OF CONCRETE

Reported by Dr. E. C. E. LORD, Petrographer, United States Bureau of Public Roads

IN PREVIOUS investigations carried on by the Bureau of Public Roads with tar and paraffin as a means for protecting concrete against alkali attack,[1] no quantitative data were obtained regarding the effect of the tar treatment on the strength of the concrete or its resistance to the action of sulphate solutions under severe service conditions.

It was planned at the outset of the investigation reported herein to carry out strength tests simultaneously on treated and untreated specimens stored out of doors in tap water at Arlington Farm, Va., and in the sulphate water of Medicine Lake, S. Dak. It was found, however, that many of the lake specimens were so severely attacked after a comparatively short period that the relative degree of protection offered by the various tar treatments could best be obtained by recording the conditions of the samples from year to year.

PREPARATION OF SPECIMENS

The concrete test specimens employed in the present investigations were 3 by 6 inch cylinders of 1:2:4 and 1:1½:3 mix, and of medium and wet consistency, made up in batches of 3 each with Potomac River sand (fineness modulus 2.60) and Potomac River gravel graded from three-fourths to one-half inch (60 per cent) and from one-half to one-fourth inch (40 per cent). The cement used passed all standard specification requirements.

The mixes were proportioned with a water-cement ratio of 1.25 for the 1:2:4 mix and 0.86 for the 1:1½:3 mix, of wet consistency (flow 180), and 1.10 for the 1:2:4 mix and 0.83 for the 1:1½:3 mix, of medium consistency (flow 140).

The specimens for compressive strength determinations were cured for 7, 28, and 90 days in damp air and 14 days in dry air for 1:2:4 concrete of medium consistency (Table 1); and 14 days in damp air and 3, 14, 28, and 60 days in dry air for 1:1½:3 and 1:2:4 mixes of medium consistency (Tables 2 and 3). Specimens of 1:1½:3 and 1:2:4 concrete, medium and wet consistency, for exposure to alkali action were cured for 14 days in damp air and 3, 7, 14, 28, and 60 days in dry air (Table 4) before receiving tar treatment.

Immediately after curing, water-gas tar conforming to grade TW–1–X of the provisional specifications for tars for use in the protection of cement concrete [2] was applied to the concrete in brush coats. The quantity of tar absorbed after each application was determined separately and calculated on a percentage basis from the weight of the untreated specimen. Following the priming coats of water-gas tar alternate batches of 3 cylinders each were given one seal coat of coal tar conforming to grade TR–1–25 of the provisional specifications.

After treatment, the samples for alkali exposure were shipped in specially prepared crates, each containing 8 batches (24 specimens), to Medicine Lake, S. Dak.[3]

The specimens for strength determination were treated in the same way and stored at Arlington Farm, Va., in a galvanized iron tank of about 400 gallons capacity, in which the water was frequently renewed.

TABLE 1.—*Compressive strength of 1:2:4 concrete of medium consistency treated with water-gas tar and coal tar after storage from one week to one year in tap water* [a]

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
Batch No.	Curing	Treatment	Tar absorbed	Penetration	Strength after stated number of weeks in water (pounds per square inch)						Per cent increase in strength after stated numberof weeks [b]					Strength rating after stated number of weeks in water				
					1	2	4	13	26	52	2	4	13	26	52	2	4	13	26	52
	Days	*No. of coats*	*P. ct.*	*Inch*												*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*
1	7 damp air, 14 dry air.	Untreated			1,968	1,998	2,201	2,630	2,588	3,053	1.5	11.8	33.7	31.5	55.1	100	100	100	100	100
2	do	4 water-gas tar	0.42	1/8	2,065	1,971	2,014	2,503	2,360	2,775	−4.6	−2.5	21.2	14.3	34.4	98.6	91.5	95.2	91.2	90.9
3	do	4 water-gas tar, 1 coal tar.	0.77	1/8	1,645	1,905	2,039	2,282	2,227	2,675	15.8	24.0	38.7	35.4	62.7	95.3	92.6	86.8	86.1	87.6
4	do	8 water-gas tar	0.93	1/4	1,672	2,014	2,144	2,033	2,183	2,679	20.5	28.2	21.6	30.6	60.2	100.8	97.4	77.3	84.4	87.7
5	do	8 water-gas tar, 1 coal tar.	1.36	1/4	1,681	2,018	1,963	2,157	1,983	2,386	20.0	16.8	28.3	18.0	41.9	101.0	89.2	82.0	76.6	78.1
6	28 damp air, 14 dry air.	Untreated							3,637	3,655					0.5				100	100
7	do	4 water-gas tar	0.20	1/16					3,476	2,895					−16.7				95.5	79.2
8	do	4 water-gas tar, 1 coal tar.	0.78	1/16					2,885	2,935					1.7				79.3	80.3
9	do	6 water-gas tar	0.62	1/8					3,228	3,010					−6.7				88.7	82.3
10	do	6 water-gas tar, 1 coal tar.	0.93	1/8					2,882	2,970					3.1				79.2	81.3
11	90 damp air, 14 dry air.	Untreated							3,395	2,923					−13.9				100	
12	do	4 water-gas tar	0.37	1/32					2,653	2,793					5.3				78.2	
13	do	4 water-gas tar, 1 coal tar.	0.69	1/32					2,792	2,795					0.1				82.2	
14	do	do	0.62	1/32					2,765	2,936					6.2				81.5	

[a] Test results indicate averages of three samples in each batch.
[b] Per cent increase in strength was calculated from values obtained at one week and six months.

[1] Public Roads, vol. 5, No. 3, May, 1924; vol. 6, No. 11, January, 1926; and vol. 8, No. 6, August, 1927.
[2] Public Roads, vol. 8, No. 6, August, 1927, p. 107.
[3] Medicine Lake is situated 18 miles northwest of Watertown, S. Dak., and is extremely rich in alkali, the sulphate content of the water varying according to the season of the year from 2.14 to 3.06 per cent magnesium sulphate and from 0.12 to 1.34 per cent sodium sulphate. Public Roads, vol. 6, No. 8, October, 1925, p. 175.

TABLE 2.—*Compressive strength of 1:1½:3 concrete, medium consistency, treated with water-gas tar and coal tar after storage from 30 to 40 months in tap water*

1	2	3	4	5	6	7	8	9	10	11	12	13
Batch No.	Crate No.	Samples tested	Curing	Treatment	Tar absorbed	Penetration	Strength after stated number of months in water, pounds per square inch				Strength deviation [1]	Strength rating
							30	34	38	40		
			Days	*No. of coats*	*P. ct.*	*Inch*					*Per cent*	*Per cent*
1	110	3	14 damp air, 3 dry air	Untreated						4,460	8.1	
2		3	do	do						4,773	1.6	
3		3	do	do						5,157	6.3	
4		3	do	do						4,927	1.5	
5		2	do	do						4,985	2.7	
Crate total and averages		14								4,852		100
6	90	3	14 damp air, 3 dry air	2 water-gas tar	0.70	1/32				4,823	−0.7	99.4
7		3	do	2 water-gas tar, 1 coal tar	1.17	1/32				4,837	−0.4	99.7
8		3	do	3 water-gas tar	.88	1/32				4,947	+1.9	101.9
9		3	do	3 water-gas tar, 1 coal tar	1.44	1/32				5,370	+10.6	110.7
10		3	do	4 water-gas tar	1.15	1/16				4,263	−12.2	87.8
11		3	do	4 water-gas tar, 1 coal tar	2.04	1/16				5,033	+3.6	103.7
12		3	do	5 water-gas tar	1.23	1/16				5,080	+4.6	104.8
13		3	do	5 water-gas tar, 1 coal tar	1.89	1/16				4 503	−7.3	92.7
Crate total and averages		24		4 coats	1.31	1/21				4,857		100.1
14	98	3	14 damp air, 14 dry air	4 water-gas tar	.62	1/16			3,930		+0.6	
15		3	do	4 water-gas tar, 1 coal tar	.90	1/16			3,963		+1.5	
16		3	do	6 water-gas tar	.76	1/8			4,070		+4.2	
17		3	do	6 water-gas tar, 1 coal tar	1.08	1/8			4,150		+6.3	
18		3	do	6 water-gas tar	.82	1/8			3,863		−1.1	
19		3	do	6 water-gas tar, 1 coal tar	1.11	1/8			4,157		+6.5	
20		3	do	6 water-gas tar	1.06	1/8			3,673		−5.9	
21		3	do	6 water-gas tar, 1 coal tar	.94	1/8			3,437		−12.0	
Crate total and averages		24		6 coats	.91	1/9			3,905			
22	102	3	14 damp air, 28 dry air	Untreated				4,660				100
23		2	do	3 water-gas tar, 1 coal tar	.96	1/16		4,650			−0.9	99.8
24		3	do	4 water-gas tar	.69	1/16		4,617			−1.6	99.1
25		3	do	4 water-gas tar, 1 coal tar	.91	1/16		4,647			−1.0	99.7
26		3	do	5 water-gas tar	.69	1/8		4,883			+4.0	104.8
27		3	do	5 water-gas tar, 1 coal tar	1.06	1/8		4,663			−0.7	100.1
28		3	do	5 water-gas tar	.65	1/8		4,690			−0.1	100.7
29		3	do	5 water-gas tar, 1 coal tar	1.04	1/8		4,697			+0.1	100.8
Crate total and averages		23		5 coats	.86	1/10		[2] 4,694				100.6
30	106	3	14 damp air, 60 dry air	Untreated			5,163					100
31		3	do	2 water-gas tar, 1 coal tar	.69	1/64	4,820				−5.2	93.3
32		3	do	3 water-gas tar	.38	1/64	5,140				+1.1	99.5
33		3	do	3 water-gas tar, 1 coal tar	.74	1/64	5,163				+1.5	100.0
34		3	do	4 water-gas tar	.47	1/32	5,277				+3.8	102.2
35		3	do	4 water-gas tar, 1 coal tar	.47	1/32	5,140				+1.1	99.5
36		3	do	4 water-gas tar	.53	1/32	4,980				−2.1	96.4
37		3	do	4 water-gas tar, 1 coal tar	.80	1/32	5,080				−0.1	98.4
Crate total and averages		24		4 coats	.58	1/41	[2] 5,086					98.7

[1] Strength deviation indicates averages of each batch compared with the crate average.
[2] Average of treated samples.

TREATED AND UNTREATED CYLINDERS SHOW LITTLE DIFFERENCE IN ULTIMATE STRENGTH AFTER STORAGE IN TAP WATER

In Table 1 are given the results of compression tests of 1:2:4 concrete cylinders of medium consistency, treated in batches of 3 cylinders each with water-gas tar and coal tar after storage from one week to one year in tap water.

It will be observed that tar absorption (columns 4 and 5) decreased as time of curing increased (column 2), and that the strength developed is at a minimum with maximum tar absorption for samples cured for a short period of time (batches 2 to 5, columns 11 and 21). The results indicate that the average increase in strength for the treated material between the ages of 1 week and 1 year is about the same as for the untreated material (batches 1 to 5, columns 12 to 16), but the strength of the treated samples after one year in water averages about 14 per cent less than that of the untreated specimens (column 21).

For samples cured 28 days in damp air and 14 days in dry air before receiving the tar treatment (batches 6 to 10) the results were inconclusive, some batches indicating a slight gain (batches 8 and 10, column 16) and others an appreciable loss in strength during the six months to one year period (batches 7 and 9, column 16). The average strength rating of these samples after one year was about 81 (column 21).

A more consistent gain in strength was indicated by specimens cured for 90 days in damp air and 14 days in dry air (batches 12 to 14, column 16) where the values obtained indicated an average gain of about 4 per cent between 6 months and 1 year in water. A retrogression in strength of over 16 per cent between 6 months and 1 year for the untreated specimens belonging to this group (batch 11, column 16) would render the computation of the strength ratings at the age of 1 year misleading, but if computed on the basis of the strength of the untreated samples after 26 weeks in water (batch 11, column 10) the strength ratings of these batches would average essentially the same at the age of 6 months as those of the foregoing group at the age of 1 year, or about 81.

From these results it may be concluded that the tar treatment retards strength development in concrete up to a period of 1 year in water; the loss in strength averaging from about 15 to 20 per cent compared with that of the untreated material.

It will be observed from the following data, however, that after longer storage the effect of the tar treatment on the strength of concrete is much less apparent.

TABLE 3.—*Compression strength of 1:2:4 concrete, medium consistency, treated with water-gas tar and coal tar after storage from 30 to 40 months in tap water*

1	2	3	4	5	6	7	8	9	10	11	12	13
Batch No.	Crate No.	Samples tested	Curing	Treatment	Tar absorbed	Penetration	Strength after stated number of months in water, pounds per square inch				Strength deviation[1]	Strength rating
							30	34	38	40		
			Days	*No. of Coats*	*P. ct.*	*Inch*					*Per cent*	*Per cent*
1	92	3	14 damp air, 3 dry air	Untreated						3,740		100
2		3	do	2 water-gas tar, 1 coal tar	1.05	1/16				3,740	+6.6	100.0
3		3	do	3 water-gas tar	1.02	1/16				3,630	+3.5	97.1
4		3	do	3 water-gas tar, 1 coal tar	1.23	1/16				3,443	−1.8	92.0
5		3	do	4 water-gas tar	1.00	1/8				3,663	+4.4	98.0
6		3	do	4 water-gas tar, 1 coal tar	1.46	1/8				3,227	−8.0	86.3
7		3	do	5 water-gas tar	1.16	1/8				3,170	−9.6	84.8
8		3	do	5 water-gas tar, 1 coal tar	1.49	1/8				3,677	+4.8	98.3
Crate total and averages		24		4 coats	1.20	1/16				[2]3,507		94.6
9	100	3	14 damp air, 14 dry air	3 water-gas tar	.64	1/8		2,977			−3.3	
10		3	do	3 water-gas tar, 1 coal tar	1.02	1/8		2,860			−7.1	
11		3	do	4 water-gas tar	.82	1/8		3,083			+0.1	
12		3	do	4 water-gas iar, 1 coal tar	1.05	1/8		3,420			+11.1	
13		3	do	5 water-gas tar	.88	1/8		3,093			+0.5	
14		3	do	5 water-gas tar, 1 coal tar	1.14	1/8		3,123			+1.4	
15		3	do	6 water-gas tar	.94	1/4		3,200			+3.9	
16		3	do	6 water-gas tar, 1 coal tar	1.15	1/4		2,873			−6.7	
Crate total and averages		24		5 coats	0.96	1/16		3,079				
17	104	3	14 damp air, 28 dry air	Untreated				3,260				100
18		3	do	3 water-gas tar, 1 caol tar	1.02	1/16		2,980			−2.9	91.4
19		3	do	4 water-gas tar	.82	1/16		2,923			−4.8	89.7
20		3	do	4 water-gas tar, 1 coal tar	1.13	1/8		3,057			−0.4	93.8
21		3	do	5 water-gas tar	.82	1/8		3,030			−1.3	93.0
22		3	do	5 water-gas tar, 1 coal tar	1.10	1/8		3,223			+5.0	98.9
23		3	do	6 water-gas tar	.91	1/4		3,107			+1.2	95.3
24		3	do	6 water-gas tar, 1 coal tar	1.35	1/4		3,170			+3.3	97.2
Crate total and averages		24		5 coats	1.02	1/4		[2]3,070				95.0
25	108	3	14 damp air, 60 dry air	Untreated			3,503					100
26		3	do	2 water-gas tar, 1 coal tar	.88	1/8	3,637				+1.6	103.8
27		3	do	3 water-gas tar	.62	1/8	3,693				+3.2	105.4
28		3	do	3 water-gas tar, 1 coal tar	1.08	1/8	3,267				−8.7	93.2
29		3	do	4 water-gas tar	.85	1/4	3,413				−4.7	97.4
30		3	do	4 water-gas tar, 1 coal tar	1.20	1/4	3,440				−3.9	98.2
31		3	do	5 water-gas tar	.94	1/4	3,603				+0.6	102.8
32		3	do	5 water-gas tar, 1 coal tar	1.14	1/4	4,010				+12.0	114.5
Crate and total averages		24		4 coats	0.96	1/16	[2]3,580					101.9

[1] Strength deviation indicate averages of each batch compared with the crate average.
[2] Average of treated samples.

In Tables 2 and 3 are recorded the results of compression tests on 1:1½:3 and 1:2:4 concrete of medium consistency after 30 to 40 months in tap water. The specimens are duplicates of those receiving the same treatment (column 5) that were sent to Medicine Lake for exposure to alkali action and have been given the same crate numbers (column 2).

The test results indicating averages of samples from each batch, as well as averages of all samples from each crate are given in columns 6 to 13. The average strength of the individual batches of 3 cylinders each receiving varying quantities of tar was found to deviate somewhat from that of the crate as a whole and this deviation, computed in percentage (column 12) has served as an index for comparison where strength ratings were not obtained.

It will be observed that the average quantity of tar absorbed reached a maximum in samples of both mixes cured for 14 days in damp air and 3 days in dry air (Table 2, batches 6 to 13, column 6, and Table 3, batches 2 to 8, column 6), and that in the richer mixes the quantity absorbed decreases with increase in time of curing until a minimum is reached at 60 days in dry air (Table 2, batches 31–37). With the leaner mixes the amount absorbed is nearly uniform beyond a curing period of 3 days in dry air (Table 3, batches 9–32). The extent of penetration varies considerably, being appreciably greater in the leaner mixes (Table 3, column 7).

These results indicate that the constituents of the hydrated cement are in an extremely tar-absorbent condition shortly after removal of the concrete from damp-air curing; but that after exposure to dry air these compounds, essentially colloidal in character, presumably undergo a change in structure and become less absorptive through loss of moisture and the action of atmospheric agencies.

Regarding the effect of the tar treatment on the strength of concrete after 2 years and more in water, the test results indicate in the case of the richer mixes (Table 2) that while a slight reduction in strength has occurred in some batches the average of the crate groups about equals that of the untreated material (Table 2, columns 8 to 13).

As regards the leaner mixes (Table 3), the results indicate that while certain batches have fallen off in strength, especially for samples cured for shorter periods in dry air (batches 6 and 7, columns 11 to 13), the average strength for each crate approaches closely to that of the untreated material (Table 3, columns 8 to 13).

From an examination of Tables 2 and 3 it is evident that no definite relationship can be found between quantity of tar absorbed and strength development. Many batches receiving a maximum quantity of tar developed a strength as great as or greater than those receiving a minimum quantity (batches 6 and 11, 16, and 19, 32 and 37, in Table 2, and batches 3 and 8, 9 and 14, 19 and 24, in Table 3).

TABLE 4.—*Exposure tests of concrete treated with water-gas tar and coal tar and stored in alkali water at Medicine Lake, S. Dak.*

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
Batch No.	Crate No.	Mix	Consistency	Curing	Treatment	Average tar absorbed	Samples attacked after stated number of months in lake											Time of failure
							6	9	12	15	18	21	24	27	33	36	39	
				Days	*No. of Coats*	*Per cent*												*Months*
1	110	1:1½:3	Medium	14 damp air, 3 dry air	Untreated					24				F [1]				27
2	90	1:1½:3	do	do	2, 3, 4, 5, water-gas tar, 1 coal tar	1.34				0				3			3	
3	91	1:1½:3	Wet	do	do	.91				0				2			9	
4	94	1:1½:3	Medium	14 damp air, 7 dry air	4, 6, 8, water-gas tar, 1 coal tar	1.35			1				9			14		
5	95	1:1½:3	Wet	do	4, 6, 7, water-gas tar, 1 coal tar	1.12			0				0			12		
6	98	1:1½:3	Medium	14 damp air, 14 dry air	4, 6, water-gas tar, 1 coal tar	.96		0				2			10			
7	99	1:1½:3	Wet	do	2, 3, 4, 5, water-gas tar, 1 coal tar	.63		6				19			F			33
8	102	1:1½:3	Medium	14 damp air, 28 dry air	3, 4, 5, water-gas tar, 1 coal tar	.86		10				22			F			33
9	103	1:1½:3	Wet	do	do	.73		3				19			F			33
10	106	1:1½:3	Medium	14 damp air, 60 dry air	2, 3, 4, water-gas tar, 1 coal tar	.58	4				16				F			33
11	107	1:1½:3	Wet	do	2, 3, 4, 5, water-gas tar, 1 coal tar	.77	24				F							18
12	111	1:1½:3	Medium	14 damp air, 7 dry air	2 water-gas tar, paraffin, and linseed oil.	.63			0				3					
13	112	1:1½:3	do	do	3, 4, water-gas tar, paraffin, and linseed oil.	.72			0				0					
14	92	1:2:4	do	14 damp air, 3 dry air	2, 3, 4, 5, water-gas tar, 1 coal tar	1.01				8				12			12	
15	93	1:2:4	Wet	do	do	.91				19				F				27
16	96	1:2:4	Medium	14 damp air, 7 dry air	4, 6, 8, water-gas tar, 1 coal tar	1.40			1				12			24		
17	97	1:2:4	Wet	do	do	1.35			6				21			F		36
18	100	1:2:4	Medium	14 damp air, 14 dry air	3, 4, 5, 6, water-gas tar, 1 coal tar	.98		24				F						21
19	101	1:2:4	Wet	do	4, 6, 5, 7, water-gas tar, 1 coal tar	1.01		24				F						21
20	104	1:2:4	Medium	14 damp air, 28 dry air	3, 4, 5, 6, water-gas tar, 1 coal tar	.99	24				F							18
21	105	1:2:4	Wet	do	do	.86	24				F							18
22	108	1:2:4	Medium	14 damp air, 60 dry air	2, 3, 4, 5, water-gas tar, 1 coal tar	.94	10				F							18
23	109	1:2:4	Wet	do	do	.93	12				F							18
24	113	1:2:4	Medium	14 damp air, 7 dry air	2 water-gas tar and paraffin	.74		21				F						21
25	114	1:2:4	do	do	3, 4, water-gas tar and paraffin	.98		24				F						21
26	115	1:2:4	do	do	2, 3, 4, water-gas tar and lubricating oil.	.97		0				F						21
27	116	1:2:4	do	do	2, 4, water-gas tar and paraffin	.77		0				F						21
28	117	1:2:4	do	do	2 water-gas tar aud paraffin	.80		0				F						21
29	118	1:2:4	do	do	4, 5, water-gas tar and paraffin	1.38	0				F							18

[1] F indicates failure of all specimens.

From the results thus far obtained it may be concluded that while the tar treatment somewhat retards the early strength development of concrete stored in water, the effect of the treatment decreases with time of storage, the average strength ultimately attained varying but little from that of the untreated material.

RESULTS OF EXPOSURE TESTS INDICATE VALUE OF WATER-GAS TAR TREATMENT UNDER PROPER CONDITIONS OF MIX, CONSISTENCY, AND CURING

The cylinders placed in Medicine Lake for exposure to alakli action were prepared and treated in the same way as those used in the strength tests except that mixes of both medium and wet consistency were employed, and in some cases (Tables 4, batches 12, 13, and 24 to 29) a small quantity (10 per cent by weight of tar) of paraffin, linseed oil or lubricating oil (viscosity 2.85, Saybolt) was incorporated with the tar.

The test results are given in Table 4, in which the indicated values represent averages of 24 specimens tested in each crate.

It may be noted that the indicated quantity of tar absorbed is of the same general order as that previously recorded for the strength tests. That is to say, samples of both mixes cured for shorter periods in dry air (batches 2 to 5 and 14 to 17, columns 5 to 7) are, in general, more absorptive than those cured for longer periods (batches 6 to 11 and 18 to 23, columns 5 to 7). The results indicate also that the dryer mixes are, on the whole, more tar absorptive than the wet mixes cured for the same length of time (batches 2 to 11, 14 to 23, columns 4 and 7).

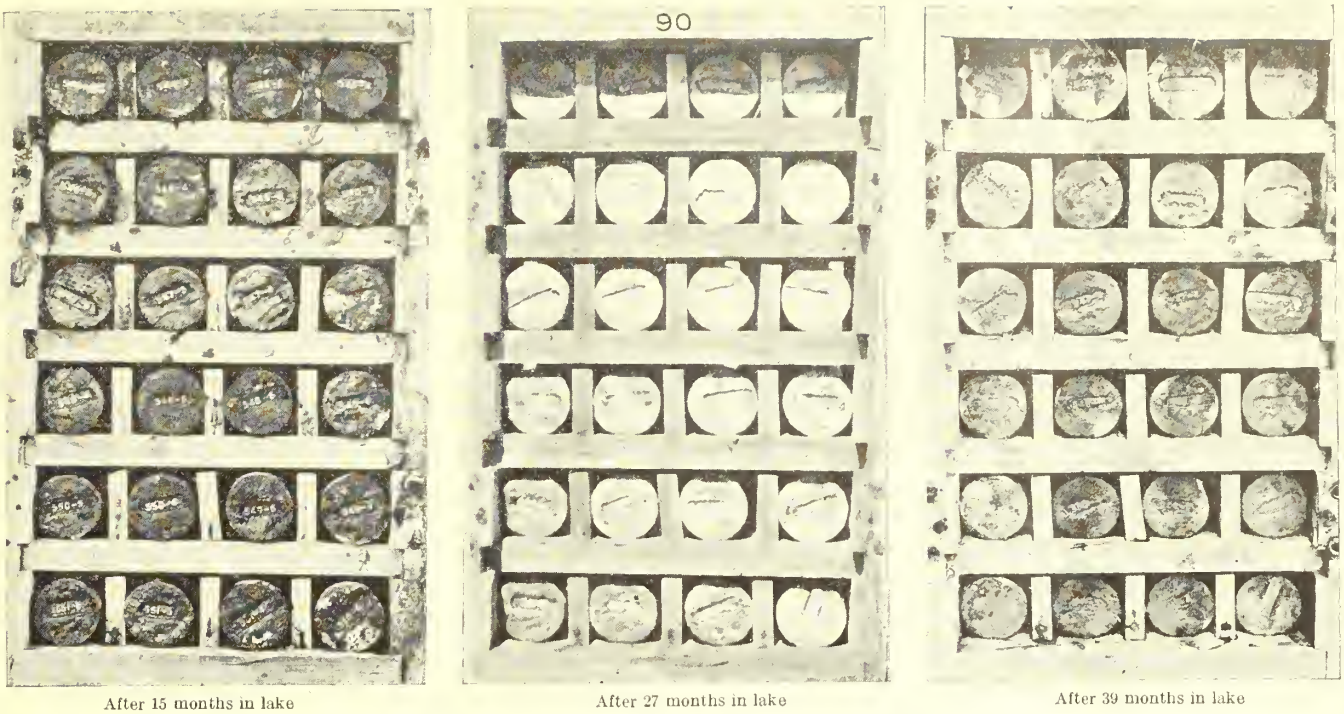
The effect of alkali action on the specimens is indicated in columns 8 to 19, in which the number of specimens attacked in each crate is given, as of the time of inspection.

It will be observed that after an exposure period of from 6 to 15 months (columns 8 to 11) all samples of untreated 1:1½:3 concrete, medium consistency, cured for 14 days in damp air and 3 days in dry air had been attacked (batch 1, column 11) while samples of the same mix, medium and wet consistency, cured in the same way and receiving from 2 to 5 coats of water-gas tar and 1 coat of coal tar were intact (batches 2 and 3, column 11). It will be noted also that while the treated samples of the same mix cured for 7 days in dry air were in a satisfactory condition after 12 months in the lake (batches 4 and 5, column 10), the protective effect of the tar treatment decreased in efficiency with prolonged curing in dry air until with 60 days' curing all samples of wet consistency were attacked after 6 months exposure (batch 11, column 8).

In the case of the leaner mixes (batches 14 to 23) the tar treatment was not so effective, but it will be noticed that samples cured for a short period in dry air before treatment, especially those of medium consistency, were in better condition after 12 to 14 months exposure (batches 14 to 17, columns 10 and 11) than those cured for a greater length of time (batches 18 to 23, columns 8 and 9).

In regard to the relative protection offered by the tar treatment on concrete of medium and wet consistency (columns 4 and 7 to 19) the test results indicate that, in general, the dryer mixes are more tar absorptive and offer greater resistance to alkali attack.

In the tabulation of the final results of the tests (column 19) it will be observed that all batches of 1:2:4 concrete receiving the tar treatment had failed within a period of from 18 to 36 months in the lake water, except those of medium consistency cured for 14 days in damp air and from 3 to 7 days in dry air (batches 14 and 16, column 19). In the case of the 1:1½:3 specimens only batches cured for 14, 28, and 60 days in dry air had failed within this period (batches 7 to 11, column 19), the remaining batches, especially the batch of medium consistency, cured for 14 days in damp air and 3 days in dry air (batch 2, column 18) being

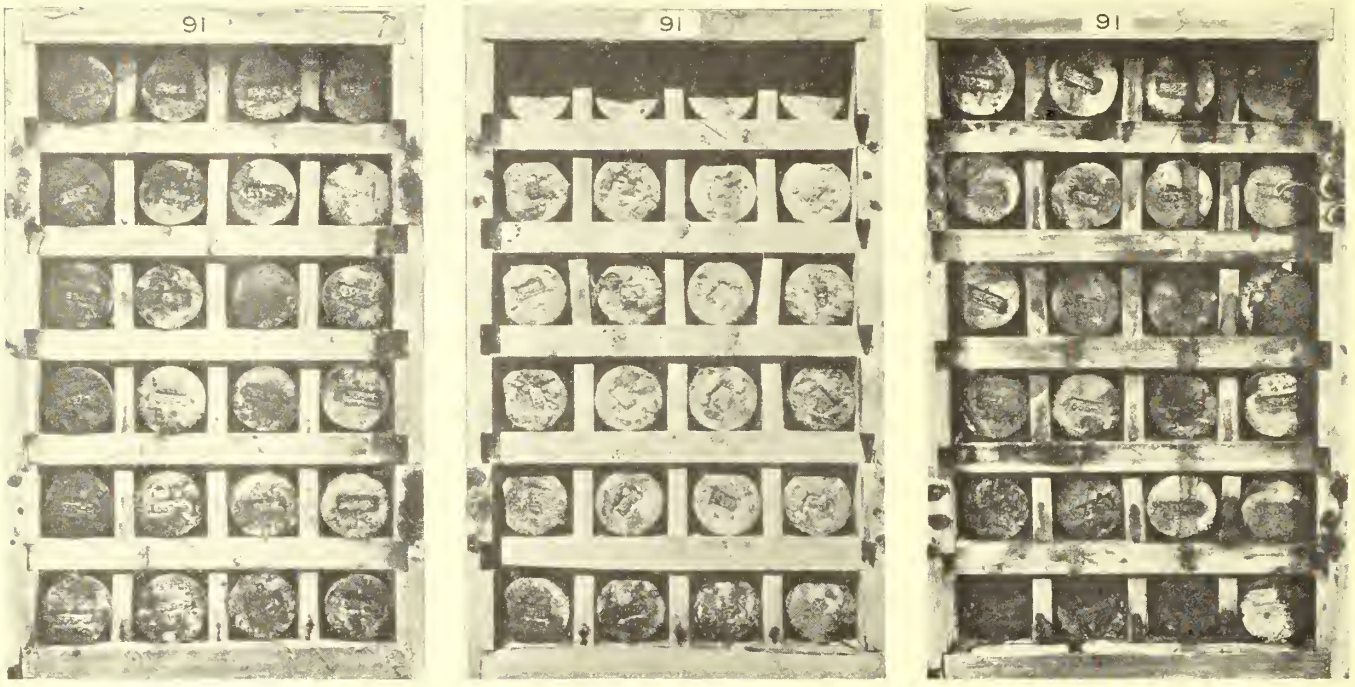


After 15 months in lake | After 27 months in lake | After 39 months in lake

FIGURE 1.—CONDITION OF 1:1½:3 CONCRETE CYLINDERS OF MEDIUM CONSISTENCY (BATCH 2, CRATE 90) AFTER 15, 27, AND 39 MONTHS IN MEDICINE LAKE, S. DAK. CURING: 14 DAYS IN DAMP AIR, 3 DAYS IN DRY AIR. TREATMENT: 2, 3, 4, AND 5 COATS OF WATER-GAS TAR AND 1 COAT OF COAL TAR

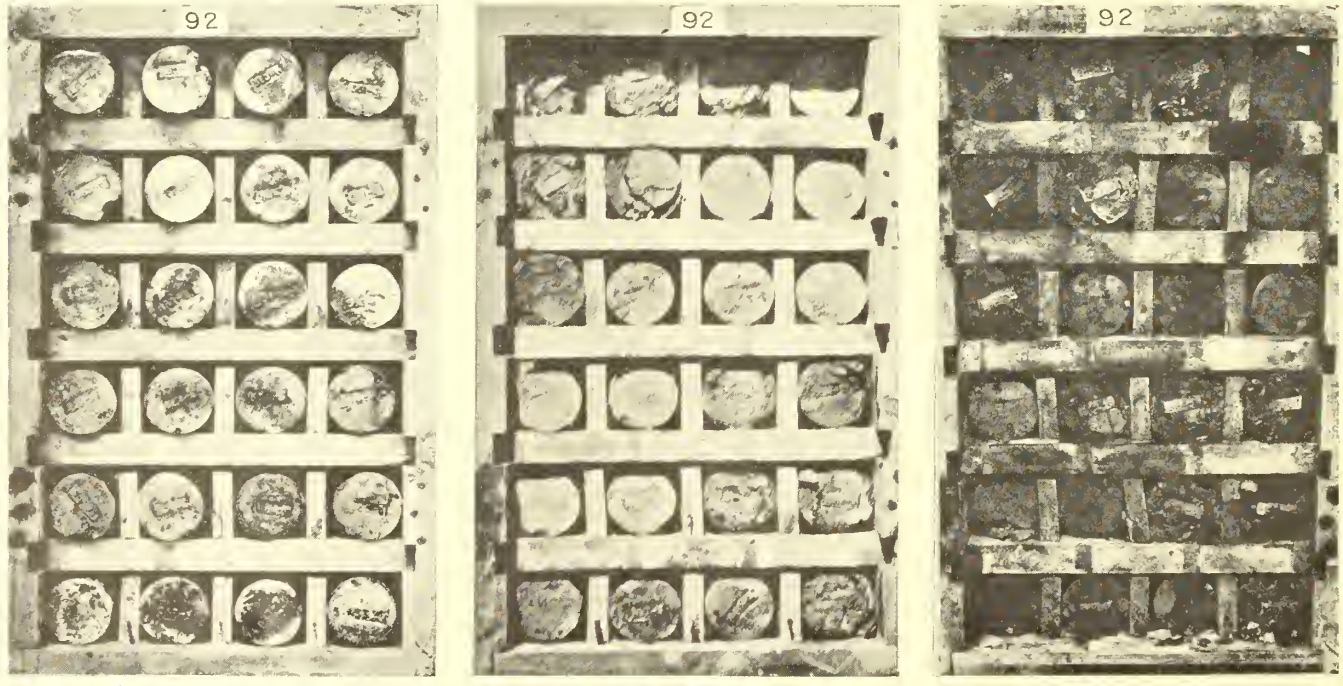
After 15 months in lake | After 27 months in lake

FIGURE 2.—CONDITION OF UNTREATED 1:1½:3 CONCRETE CYLINDERS OF MEDIUM CONSISTENCY (BATCH 1, CRATE 110) AFTER 15 AND 27 MONTHS IN MEDICINE LAKE, S. DAK. CURING SAME AS IN FIGURE 1



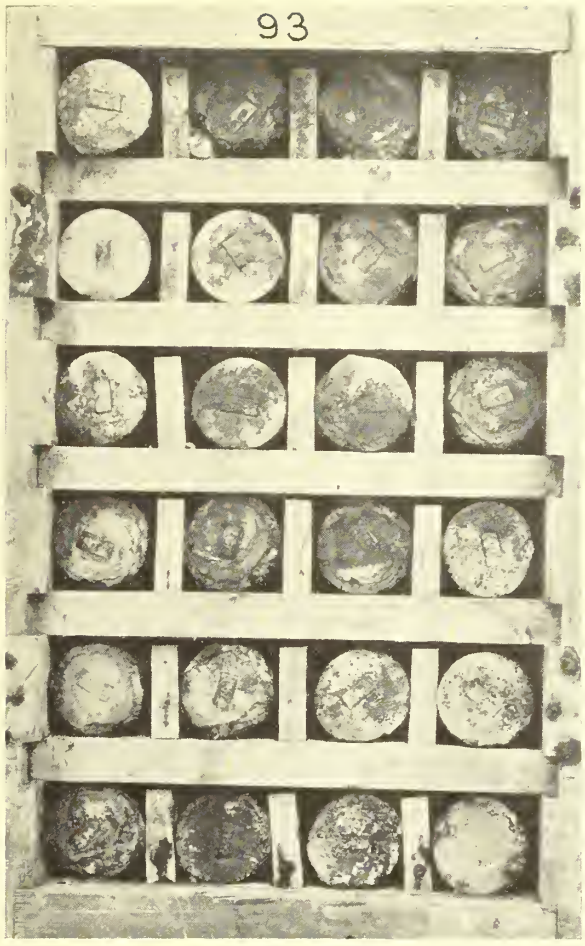
After 15 months in lake | After 27 months in lake | After 39 months in lake

FIGURE 3.—CONDITION OF 1:1½:3 CONCRETE CYLINDERS OF WET CONSISTENCY (BATCH 3, CRATE 91) AFTER 15, 27, AND 39 MONTHS IN MEDICINE LAKE, S. DAK. CURING AND TREATMENT SAME AS IN FIGURE 1



After 15 months in lake | After 27 months in lake | After 39 months in lake

FIGURE 4.—CONDITION OF 1:2:4 CONCRETE CYLINDERS OF MEDIUM CONSISTENCY (BATCH 14, CRATE 92) AFTER 15, 27, AND 39 MONTHS IN MEDICINE LAKE, S. DAK. CURING AND TREATMENT SAME AS IN FIGURE 1



After 15 months in lake

FIGURE 5.—TEST SHOWING CONDITION OF 1:2:4 CONCRETE CYLINDERS OF WET CONSISTENCY (BATCH 15, CRATE 93) AFTER 15 MONTHS IN MEDICINE LAKE, S. DAK. CURING AND TREATMENT SAME AS IN FIGURE 1. THESE SPECIMENS FAILED AFTER 27 MONTHS IN THE LAKE

in good condition. The condition of the cylinders in this batch, after 15, 27, and 39 months exposure, is shown in Figure 1.

Regarding the addition of paraffin and lubricating oil to the tar, the results indicate that these mixtures are ineffectual in retarding alkali attack, samples thus treated having failed within 18 to 21 months in the lake (batches 24 to 29, column 19). The addition of linseed oil, however, to the tar and paraffin mixture appears to be beneficial when applied to the richer mixes, as the samples receiving this treatment (batches 12 and 13, column 14) were practically unattacked after 24 months in the lake (see fig. 6). Linseed oil was not used with the leaner mixes.

PHOTOGRAPHS SHOW CONDITION OF SAMPLES

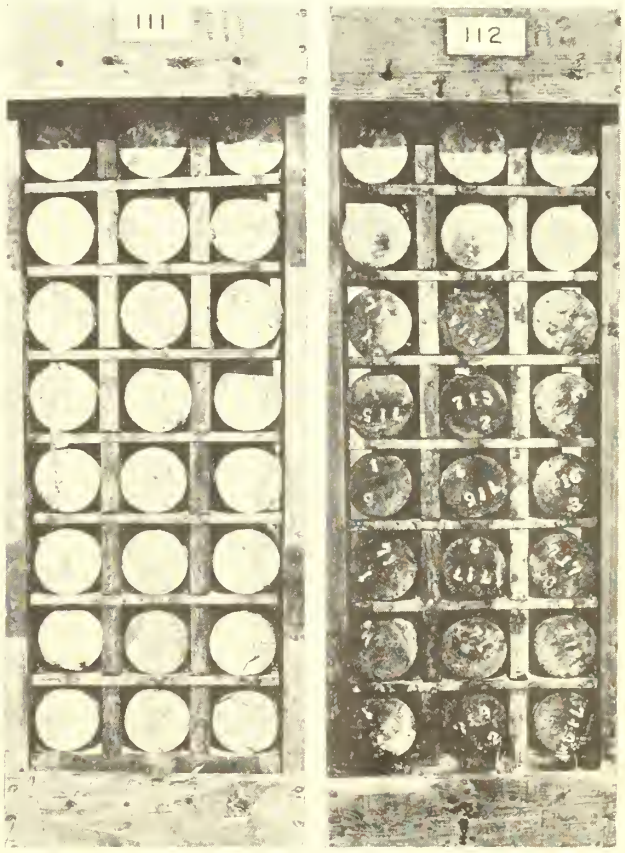
The condition of the samples after exposure to alkali action may be seen from photographs of the crates taken immediately after their removal from the lake at the time the yearly inspections were made. It should be stated that the samples receiving the minimum quantity of tar were placed in the upper tier in the crates and those receiving progressively larger quantities in the ones below.

The condition of untreated 1:1½:3 concrete of the same consistency and curing as the treated samples in crate No. 90 (fig. 1), after 15 and 27 months in the lake is shown in Figure 2.

It will be noticed that where alkali action had progressed to an appreciable extent the specimens attacked were about evenly distributed throughout the crates (fig. 3, right, fig. 4, center and right, and fig. 5, batches 3, 14, and 15, crates 91, 92, and 93) indicating that the efficiency of the treatment can not be estimated with certainty by the quantity of tar absorbed nor by the addition of a coal tar seal coat. It will be seen later on that the degree of protection is influenced to a greater extent by richness and consistency of mix and method of curing the concrete previous to the treatment than by the quantity of tar applied.

This is brought out by comparing the 1:1½:3 cylinders of medium consistency cured for 14 days in damp air and 3 days in dry air (fig. 1, batch 2, crate 90) with those of wet consistency cured in the same manner (fig. 3, batch 3, crate 91). In crate 90 all the cylinders were in good condition after 39 months in the lake except 3 in the upper tier which had received only 2 coats of water-gas tar (12 grams per cylinder), whereas in crate 91 (fig. 3, right) at least 9 cylinders which had received as much as 16 grams of water-gas tar and 5 grams of coal tar each were badly attacked.

The protective effect of the tar treatment on 1:1½:3 concrete decreases with time of curing in dry air, and it has been shown that all samples cured for longer than 14 days in dry air before treatment failed after 18 to 33 months in the lake water. (Table 4, batches 8 to 11, column 19.)



Batch 12, Crate 111. Batch 13, Crate 112

Curing: 14 days in damp air, 7 days in dry air
Treatment: Batch 12, 2 coats of water-gas tar containing 10 per cent paraffin and 10 per cent linseed oil; batch 13, 3, and 4 coats of water-gas tar containing 10 per cent paraffin and 10 per cent linseed oil

FIGURE 6.—CONDITION OF 1:1½:3 CONCRETE CYLINDERS OF MEDIUM CONSISTENCY, TREATED WITH WATER-GAS TAR CONTAINING PARAFFIN AND LINSEED OIL, AFTER 24 MONTHS IN MEDICINE LAKE, S. DAK.

In the case of the 1:2:4 concrete the conditions are similar to those of the richer mixes although the effect of alkali action is much more evident. This fact may be observed by comparing specimens of medium consistency cured 14 days in damp air and 3 days in dry air (fig. 4, batch 14, crate 92) with samples of the same mix and curing but of wet consistency (fig. 5 batch 15, crate 93). While the latter samples were seriously attacked after 15 months' exposure and had failed completely within 27 months, about one-half the samples of medium consistency were intact at the end of this period (fig. 4, center). These same samples were still in good condition at the end of 39 months (fig. 4, right). As indicated in the table, all specimens of 1:2:4 concrete cured for more than 7 days in dry air had failed within 18 and 21 months' exposure to the lake water. (Table 4, batches 18 to 23, column 19.)

A possible benefit derived by the addition of paraffin and linseed oil to the water-gas tar when applied to 1:1½:3 concrete is indicated in Figure 6 (crates 111 and 112, batches 12 and 13). It should be recalled, however, that all samples of 1:2:4 concrete treated with paraffin and lubricating oil had failed after 22 months' exposure (Table 4, batch Nos. 24 to 29, column 19), and it seems doubtful if the addition of linseed oil to the tar would be of great permanent benefit.

CONCLUSIONS SUMMARIZED

The results of the present investigations lead to the following conclusions:

1. The compressive strength of concrete cured for varying periods in damp and dry air and treated with 4 or more coats of water-gas tar and 1 coat of coal tar is somewhat lowered during early stages of hardening under water, but this loss is ultimately almost fully regained.

2. During the early stages the strength decreases rather uniformly with increasing quantities of tar absorbed, but when ultimate strength is attained the effect of the tar is negligible.

3. The protection against alkali action afforded concrete by the tar treatment is influenced by the cement content, method of curing and consistency of mix, being most effective in rich mixes of medium consistency cured for a minimum length of time in dry air.

4. Concrete of wet consisteney is less tar-absorptive and offers lower resistance to alkali attack than concrete of medium consistency cured in the same manner, although no positive relation is shown to exist between the amount of tar absorbed and the resistance to alkali attack.

5. The protection to concrete afforded by the water-gas tar treatment, under the severe conditions imposed in this investigation, was increased only to a limited extent by the application of a nonpenetrable surface coat of coal tar.

6. Concrete of high cement content, cured for a limited period in dry air and treated with water-gas tar is capable of offering appreciable resistance to alkali attack under the conditions imposed in the foregoing investigation.

(Continued from page 108)

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID AND EMERGENCY ROAD CONSTRUCTION

AS OF

MAY 31, 1931

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION						BALANCE OF FEDERAL AID FUNDS AVAILABLE FOR NEW PROJECTS	BALANCE OF EMERGENCY ADVANCE FUNDS AVAILABLE FOR NEW PROJECTS
		ESTIMATED TOTAL COST	FEDERAL AID ALLOTTED	EMERGENCY ADVANCE FUND	MILEAGE INITIAL	MILEAGE *STAGE	MILEAGE TOTAL	ESTIMATED TOTAL COST	FEDERAL AID ALLOTTED	EMERGENCY ADVANCE FUND	MILEAGE INITIAL	MILEAGE *STAGE	MILEAGE TOTAL		
ALABAMA	2,113.5	$ 7,060,353.62	$ 3,447,247.65	$ 1,370,863.51	208.1	117.0	325.1	$ 585,899.52	$ 293,449.73	$ 277,865.45	28.7	5.1	34.8	$ 3,355,041.20	$ 49,815.04
ARIZONA	853.3	5,151,196.04	3,422,908.84	999,074.98	200.8	182.9	383.8	1,484,583.29	1,107,244.85	155,886.98	62.7	30.3	93.0	159,568.58	1,199.89
ARKANSAS	1,746.4	9,450,679.29	4,030,656.68	1,260,167.00	220.8	92.1	312.9	1,492,194.39	718,448.26	128,000.00	38.7	15.9	53.5	448,154.04	
CALIFORNIA	1,958.4	11,744,703.79	4,977,232.15	1,700,711.71	292.7	44.5	337.2	2,541,841.88	1,205,048.85	648,773.52	55.5	55.1	120.6	1,093,361.55	758,747.77
COLORADO	1,328.4	5,218,229.84	2,784,233.03	987,060.51	160.2	89.0	249.2	1,088,611.68	607,750.27	306,108.40	54.7	21.3	76.0	2,183,553.50	214,572.09
CONNECTICUT	257.3	4,643,883.48	1,755,735.33	520,491.00	43.8		43.9	775,572.90	244,181.06		10.8		10.9		
DELAWARE	306.0	1,041,247.28	519,888.13	400,000.00	52.8		52.9	148,852.50	74,826.25		5.9		5.9	88,702.48	
FLORIDA	540.0	6,375,037.02	2,538,240.48	562,161.36	142.3		142.3	1,148,882.72	507,584.95	618,398.39	33.2		33.2	1,515,531.14	7,878.25
GEORGIA	2,756.2	8,846,394.45	4,296,367.99	1,880,127.33	318.8	101.8	418.8	2,848,258.37	1,307,709.43	197,196.52	106.6	34.1	140.7	1,060,913.36	673.06
IDAHO	1,289.3	3,570,306.01	2,043,140.06	603,149.65	238.4	21.9	260.3	1,088,237.58	584,147.14	374,155.48	87.9	23.0	90.8	591,278.37	30,729.87
ILLINOIS	2,218.7	24,224,950.88	11,102,587.55	1,751,000.00	711.2	43.4	754.8	8,587,588.68	3,858,112.12	1,644,000.00	281.1	4.7	285.8	1,499,724.64	5,115.00
INDIANA	1,579.0	8,444,177.22	4,623,370.88	1,148,797.10	282.4		262.4	3,414,004.11	1,555,560.92	730,937.83	107.5		107.5	565,897.04	165,194.07
IOWA	3,153.0	7,406,514.83	3,136,700.81	1,963,389.00	197.2	73.3	240.5	535,822.75	218,775.38	153,000.00		20.2	20.2	4,069.10	
KANSAS	3,120.4	7,581,353.77	3,919,068.59	952,453.38	446.8	38.8	485.6	2,898,251.90	1,381,113.94	415,507.95	148.8	40.1	188.7	685,061.21	785,918.35
KENTUCKY	1,549.3	8,724,280.50	2,950,303.88	841,985.71	283.2	90.8	354.0	3,187,235.48	858,551.38	515,990.08	84.8	74.2	139.1	207,441.93	46,759.21
LOUISIANA	1,422.9	8,935,142.75	4,115,439.52	1,132,343.87	269.8	17.3	286.9	122,208.27	68,515.39	10,000.00	5.1	.8	5.7	216,258.51	5,583.13
MAINE	593.4	3,230,988.21	1,142,584.77	426,544.24	58.7		59.7	2,117,279.53	885,158.13	289,254.79	64.7		64.7	811,458.13	
MARYLAND	704.4	1,308,803.58	586,755.89	501,088.38	29.0	4.3	32.3	699,210.72	348,606.36	150,136.41	35.3		35.3	143,489.16	27,529.23
MASSACHUSETTS	724.3	8,270,967.53	1,979,587.88	1,141,480.00	88.8		88.8	1,282,505.26	420,709.94		14.8		14.8	2,019,371.02	
MICHIGAN	1,804.8	9,874,682.80	4,029,607.21	848,000.00	265.9	22.4	288.3	1,939,851.96	796,268.00	386,000.00	54.3	15.7	70.0	2,804,888.98	1,289,382.00
MINNESOTA	4,107.4	3,925,800.10	1,914,177.75	845,000.00	44.6	164.4	209.0	5,034,424.68	2,175,498.19	1,284,993.00	35.3	187.9	203.2	21,518.47	120,000.00
MISSISSIPPI	1,772.7	3,806,886.27	1,872,537.77	1,351,844.54	158.5	80.5	239.0	602,078.19	251,039.06	42,343.18	17.2	15.4	33.5	3,851,023.96	40,548.30
MISSOURI	2,628.9	9,149,107.82	3,562,838.86	1,328,296.68	217.5	93.8	281.3	2,508,014.03	1,060,797.97	1,051,155.83	88.3	29.5	97.9	840,461.83	147,371.38
MONTANA	1,841.1	11,705,509.88	8,689,007.79	1,549,535.87	900.5	92.8	993.3	1,291,553.90	726,068.18	122,394.13	83.7	94.3	178.0	2,013,289.58	
NEBRASKA	3,815.4	8,667,914.12	4,059,638.34	1,432,333.74	232.4	204.8	437.2	2,222,492.58	1,045,506.62	275,597.28	75.7	84.4	180.1	1,077,770.23	
NEVADA	1,184.1	2,315,183.89	1,583,829.83	649,296.82	74.2	155.5	230.7	1,200,015.54	726,439.96	429,490.07	33.8	121.3	155.1	350,347.84	70,851.11
NEW HAMPSHIRE	383.9	715,470.36	281,215.41	58,215.83	10.5		10.8	491,074.31	204,099.02	208,783.17	13.7	1.0	14.7	252,439.51	135,000.00
NEW JERSEY	555.2	5,209,545.44	1,794,969.33	1,103,770.59	74.7		74.7	144,715.24	3,420.00		.2		.2	1,039,898.43	4,035.41
NEW MEXICO	1,909.4	8,897,687.96	4,185,311.84	1,237,875.95	260.5	149.1	409.5	195,139.42	123,712.23	27,456.79	.2	10.5	10.8	151,097.44	37,955.28
NEW YORK	2,587.9	37,240,368.73	12,478,253.00	3,233,568.00	603.8	7.0	510.8	9,758,396.00	3,580,448.00	295,000.00	153.9		153.9	58,212.84	522,000.00
NORTH CAROLINA	1,864.2	9,284,310.41	3,029,281.48	1,495,810.48	188.9	37.8	237.8	1,423,111.91	711,282.54	406,527.78	40.9	38.8	77.8	1,571,546.87	5,000.00
NORTH DAKOTA	4,377.8	4,098,869.01	2,077,144.19	865,008.46	469.0	358.0	825.0	1,305,201.75	852,500.70	314,781.59	121.5	259.8	391.3	1,247,030.57	118,741.85
OHIO	2,548.1	14,342,823.95	4,534,965.20	1,279,206.55	213.3	17.7	231.0	5,702,551.70	1,705,713.81	1,713,332.46	82.5	19.3	101.9	1,918,069.66	5,000.00
OKLAHOMA	1,839.8	7,736,746.31	3,865,012.99	1,518,857.26	295.8	124.5	420.3	1,398,060.13	763,505.71	407,493.74	50.8	18.2	69.0	.02	
OREGON	1,259.5	7,381,089.21	4,115,307.31	795,397.31	224.2	83.4	317.8	1,359,755.96	878,312.31	520,008.58	77.0	1.8	78.8	396,256.10	
PENNSYLVANIA	2,659.3	9,408,171.07	3,479,647.87	882,211.09	115.7		115.7	5,381,299.87	2,508,570.90	2,382,308.75	172.9		172.9	1,821,755.06	288,423.15
RHODE ISLAND	209.8	2,838,304.80	1,118,710.90	400,000.00	47.8		47.8	91,465.74	45,732.88		.1		.1	72,870.99	
SOUTH CAROLINA	1,840.1	5,844,825.26	2,875,858.82	1,114,535.00	83.3	164.9	248.2							204,800.31	
SOUTH DAKOTA	3,718.2	5,532,393.30	3,069,749.40	1,153,888.39	436.4	181.2	596.5	468,287.54	231,404.81	127,575.51	25.1	35.2	60.3	857,861.51	52,609.10
TENNESSEE	1,458.2	4,527,847.60	2,238,445.06	1,656,863.15	191.8	13.4	205.3	478,499.59	239,249.83	48,929.00	34.8		34.9	1,659,028.31	36,089.64
TEXAS	5,984.5	18,849,250.75	8,478,317.80	4,445,451.02	800.2	291.2	1,091.4	3,347,992.78	1,593,384.36	642,528.88	130.7	47.5	178.3	3,985,839.70	
UTAH	1,011.2	2,148,894.83	1,298,783.75	503,606.63	115.5	63.0	179.6	582,012.73	300,744.11	252,750.88	27.4	88.8	115.3	871,348.75	88,092.97
VERMONT	301.5	1,230,278.38	504,939.26	311,064.10	23.5	4.8	28.3	184,608.31	91,496.71	50,275.93	7.1		7.1	22,073.06	28,659.87
VIRGINIA	1,518.8	5,158,087.87	2,797,238.02	1,446,899.23	256.9	47.4	304.3	430,452.46	212,582.35	43,200.00	18.2	.5	19.8	729,721.02	15,402.77
WASHINGTON	1,001.0	5,120,210.06	2,290,609.05	975,854.93	170.7	18.8	190.5	859,509.85	353,885.90	255,842.79	15.8	12.5	29.3	1,197,006.02	37,235.28
WEST VIRGINIA	758.2	4,635,470.21	1,789,818.30	409,585.46	109.6	12.5	122.0	1,575,547.62	518,272.31	429,359.48	34.5	12.9	47.4	319,715.55	36,399.05
WISCONSIN	2,460.8	5,189,172.56	2,672,575.23	1,095,000.00	155.7	28.5	194.2	3,475,355.14	1,504,480.83	824,500.00	84.8	51.0	135.8	215,115.71	72,910.00
WYOMING	1,682.5	3,840,757.41	2,404,134.25	589,862.90	270.0	172.3	442.3	587,366.57	347,077.89	272,896.78	38.5	93.8	132.4	482,138.01	156,523.32
HAWAII	46.3	1,453,155.03	597,597.83	375,983.99	42.4		42.4	82,324.99	22,869.00		1.5		1.5	1,581,318.77	24,015.01
TOTALS	88,783.0	355,145,751.87	158,144,532.70	54,974,327.69	11,228.5	3,475.7	14,705.2	90,221,263.38	39,495,015.02	18,428,987.83	2,707.3	1,555.3	4,263.5	48,744,393.25	5,445,164.72

(*) THE TERM STAGE CONSTRUCTION REFERS TO ADDITIONAL WORK DONE ON PROJECTS PREVIOUSLY IMPROVED WITH FEDERAL AID. IN GENERAL, SUCH ADDITIONAL WORK CONSISTS OF THE CONSTRUCTION OF A SURFACE OF HIGHER TYPE THAN WAS PROVIDED IN THE INITIAL IMPROVEMENT.

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A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 12, NO. 5 ▽ JULY, 1931

A FOUR-LANE HIGHWAY IN PENNSYLVANIA

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

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THE SUBGRADE SOIL CONSTANTS, THEIR SIGNIFICANCE, AND THEIR APPLICATION IN PRACTICE

Reported by C. A. HOGENTOGLER, Senior Highway Engineer, A. M. WINTERMYER, Assistant Highway Engineer, and E. A. WILLIS, Assistant Highway Engineer, U. S. Bureau of Public Roads

PART II: A DISCUSSION OF THE SOIL CONSTANTS AND THE SOIL IDENTIFICATION CHART

THE FIRST part of this series of articles, published in the June, 1931, issue of PUBLIC ROADS, dealt with the five major physical properties of soils, internal friction, cohesion, compressibility, elasticity, and capillarity, and their relation to subgrade performance. It was shown that the presence of these properties in varying degrees depends very largely on the soil constituents, although the state in which a soil exists is a factor which can not be neglected. A method was outlined for classifying soils according to the predominance of certain constituents in them, and therefore, in a general way, according to their physical characteristics. The present report (Part II) discusses the test constants which serve to identify the constituents of soils and their resulting properties; and the soil identification chart, which is a convenient means of analyzing the data given by the laboratory tests. Part III describes the manner in which the soil identification chart is used in actual practice.

MECHANICAL ANALYSIS LIMITED AS AN INDICATOR OF SUBGRADE EFFICIENCY

From the information given in Part I it is clear that a knowledge of the size of soil particles does not furnish a complete identification of subgrade characteristics. Indeed, the mechanical analysis is very much limited in its ability to disclose accurately the size of the smaller soil particles, which must be determined by means of some method of sedimentation instead of by sieves. As a consequence, for grains smaller than about 0.074 millimeter (No. 200 sieve) in diameter, the grain diameters given by the mechanical analysis instead of being the diameters of the grains tested are the diameters of spheres which, according to Stokes law, (*20*), settle in water at a rate equal to that of the particles of soil being analyzed. A further inaccuracy is introduced by the fact that the rate at which small soil particles settle depends upon the extent to which the soil is dispersed; and this in turn depends upon both the method of agitation and the chemical reagent used in dispersing the soil.

Thus, for instance, a soil which, according to the mechanical analysis, contains 50 per cent of clay, does not necessarily contain 50 per cent of particles having diameters smaller than 0.005 millimeters. The correct interpretation of the test is that 50 per cent of the particles settle through water at a rate equal to that of spherical particles not exceeding 0.005 millimeters in diameter.

Figure 34,[7] which shows photomicrographs of soil suspensions at different periods of time after dispersion, discloses how very much the shape of soil particles is likely to vary from the spherical.

Table 4, furnished by L. B. Olmstead, L. F. Alexander, and H. E. Middleton (*20*), illustrates how the type of dispersing agent influences the silt and the clay contents furnished by the mechanical analysis. As shown in this table, the value obtained for Houston black clay may be 45 per cent or 64 per cent, depending on whether sodium hydroxide or sodium oxalate is used as a dispersing agent. Furthermore, the clay fraction depends upon the degree of agitation used during dispersion. Too little fails to separate the particles sufficiently, causing the indicated clay content to be too small. Too much agitation not only separates the particles but may also break them into smaller pieces, thus causing the indicated clay content to be too large.

TABLE 4.—*Yield of clay obtained by use of various dispersing agents* [1]

Soil type and source	Dispersing agent							
	Ammonia		Sodium hydroxide		Sodium carbonate		Sodium oxalate	
	Percentage of clay particles obtained							
	0.005 mm.	0.002 mm.	0.005 mm.	0.002 mm.	0.005 mm.	0.002 mm.	0.005 mm.	0.002 mm.
Wabash silt loam (Nebraska)	30.7	24.1	33.2	28.5	33.1	28.3	34.5	29.6
Houston black clay (Texas)	38.4	26.4	44.5	30.6	61.3	53.5	63.8	53.8
Carrington loam (Iowa)	7.8	6.4	24.9	21.8	23.4	19.9	24.7	22.9

[1] Average of duplicate determinations.

It should be remembered also that when grain size is computed from a knowledge of rate of settlement of particles in water, it is assumed that all of the grains in the soil mass being tested possess equal specific gravities, and this is not necessarily true.

The fact that mechanical analysis is inadequate to identify the characteristics of subgrade soils is illustrated by the gradings typical of the uniform subgrade groups, which are given in the following paragraphs. The two terms, "effective size" and "uniformity coefficient," used in this discussion, are defined as follows:

The effective size is the maximum size of the smallest 10 per cent, by weight, of the soil particles. On a soil accumulation curve (fig. 35) the value of the effective size is given by the abscissa of the point on the curve having the ordinate 10 per cent.

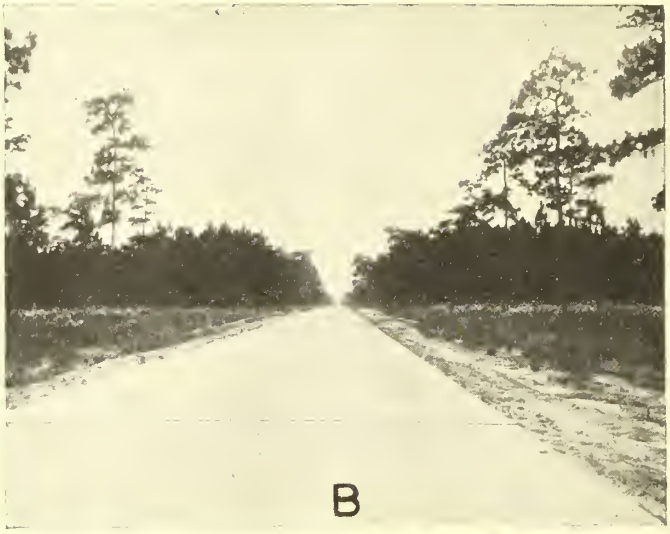
The uniformity coefficient is the ratio between the maximum size of the smallest 60 per cent, by weight, of the soil particles, and the effective size. Its value may be obtained from a soil accumulation curve by computing the ratio between the abscissa of a point whose ordinate is 60 per cent and the abscissa of a point whose ordinate is 10 per cent.

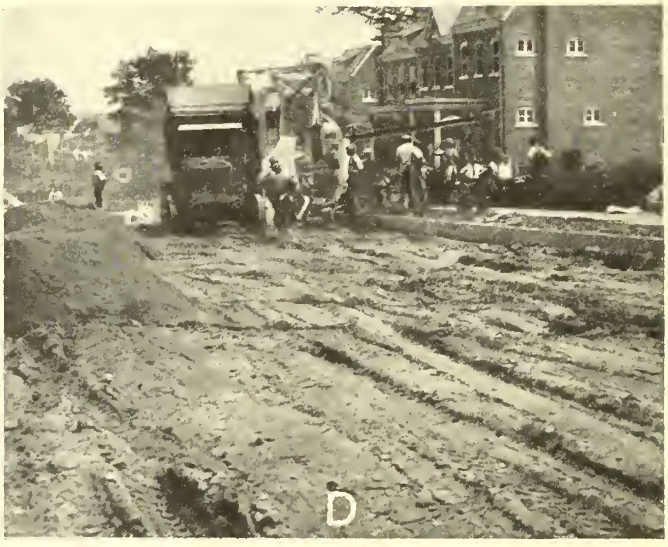
The typical gradings are as follows:

Group A–1 subgrade.—Material retained on the No. 10 sieve, not more than 50 per cent. The soil mortar, that fraction passing the No. 10 sieve, composed as follows: Clay, 5 to 10 per cent; silt, 10 to 20 per cent; total sand, 70 to 85 per cent; and coarse sand (retained on the No. 60 sieve) 45 to 60 per cent. Effective size approximately 0.01 millimeter and uniformity coefficient greater than 15.

The soil accumulation curve of Figure 35 shows graphically the average grading of stable soil mortars. In this case the

[7] The numbers of figures, tables, footnotes, and equations are consecutive with those of Part I of this report (Public Roads, June, 1931). Italic numerals in parentheses refer to the bibliography given at the end of Part I.





EXAMPLES OF SUBGRADE CLASSIFICATION AND PAVEMENTS LAID ON DIFFERENT TYPES OF SUBGRADE. A, BASE COURSE BEING CONSTRUCTED OF GROUP A–3 MATERIAL; B, CONCRETE PAVEMENT LAID ON GROUP A–3 SUBGRADE IN FLORIDA; C, BITUMINOUS MACADAM WEARING SURFACE ON GROUP A–4 BASE. D, GROUP A–5 SUBGRADE SHOWING REBOUND AFTER BEING ROLLED. E, EXCELLENT CONCRETE PAVEMENT CONSTRUCTED ON GROUP A–5 SUBGRADE. F, SURFACE TREATED MACADAM CONSTRUCTED ON AN EXCELLENT A–6 SUBGRADE.

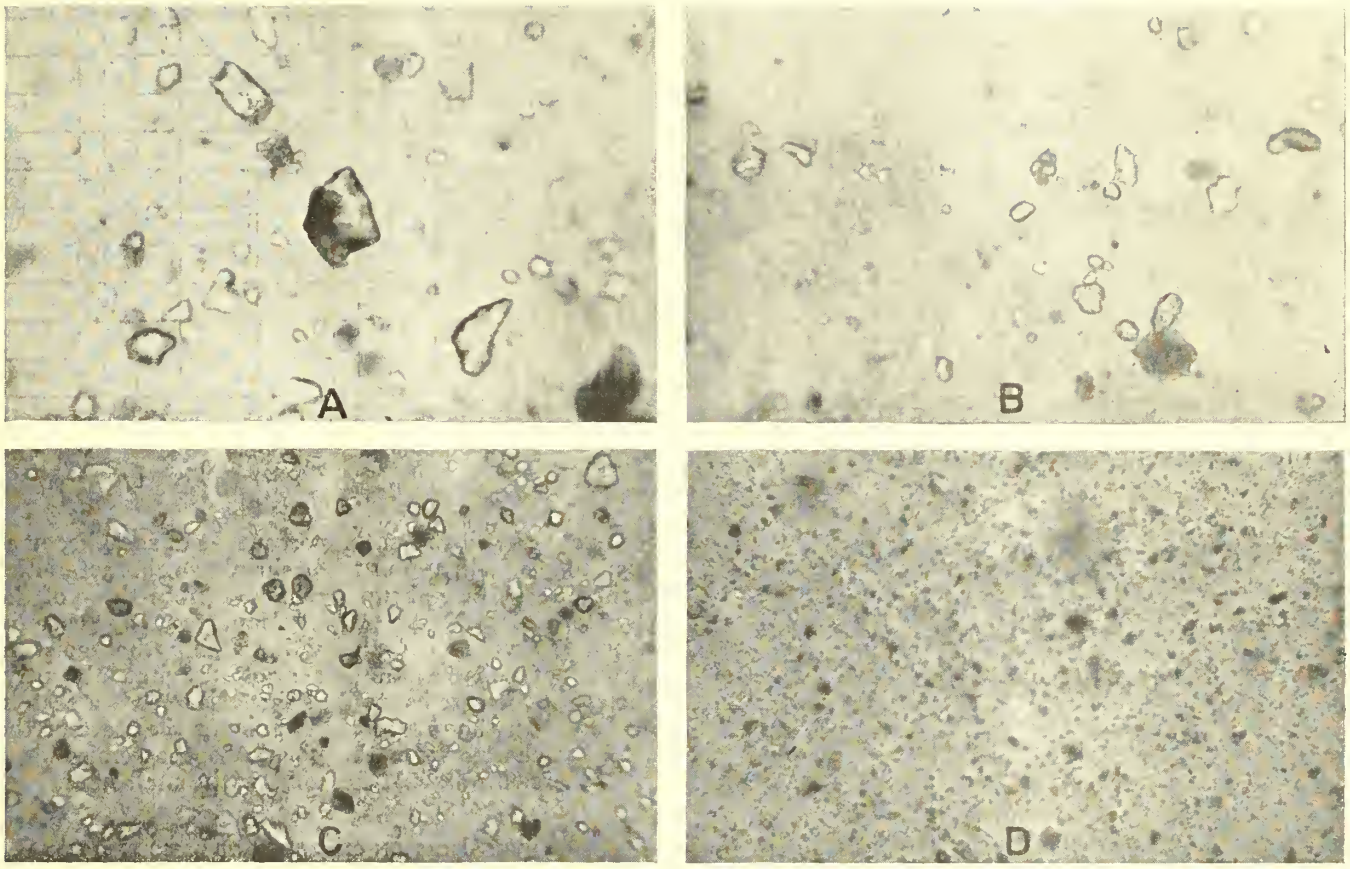


FIGURE 34.—PHOTOMICROGRAPHS OF SOIL SUSPENSION AT DIFFERENT PERIODS OF TIME AFTER DISPERSION: A, AFTER 1 MINUTE; B, AFTER 2 MINUTES; C, AFTER 5 MINUTES; D, AFTER 15 MINUTES. NOTE REDUCTION IN SIZE OF PARTICLES IN SUSPENSION AS TIME OF SEDIMENTATION INCREASES

effective size is 0.01 millimeter and the uniformity coefficient is $\frac{0.43}{0.01}$, or 43.

Group A–2 subgrade.—Not less than about 55 per cent of sand in the soil mortar.

Group A–3 subgrade.—Effective size not likely to be less than 0.10 millimeter.

Group A–4 subgrade.—Likely to contain sand in amount less than 55 per cent.

Group A–5 subgrade.—Same as group A–4.

Group A–6 subgrade.—Likely to contain more than 30 per cent clay.

Group A–7 subgrade.—Same as group A–6.

Group A–8 subgrade.—Grading not significant.

From the facts given in the preceding discussion it is plain that mechanical analysis gives only approximately the diameter of soil particles of small size, and that the size of grain, even if accurately known, is a very imperfect criterion of subgrade soil characteristics. In order to identify these characteristics it is necessary to employ constants which disclose the degree to which particular physical properties are present in a given soil.

SIGNIFICANCE OF TEST CONSTANTS DISCUSSED

Among the constants which have been suggested as aids in identifying the important subgrade properties are the liquid limit, the plastic limit, the plasticity index, the shrinkage limit, the centrifuge moisture equivalent, the field moisture equivalent, the shrinkage ratio, the volumetric change, and the lineal shrinkage. In Table 5 are listed values of these test constants for a group of representative subgrade soil constituents. In the tests from which the values given in Table 5 were derived the following materials were used:

Sand.—Potomac River sand; that fraction passing the No. 20 and retained on the No. 100 sieve.

Silt.—Silty sand soil obtained in Rock Creek Park, District of Columbia.

Clay.—Yaguajay clay (clay about 70 per cent and silt about 30 per cent) from Cuba. Furnished by H. H. Bennett, United States Bureau of Chemistry and Soils.

Colloids.—Bentonite. According to C. S. Ross and C. V. Shannon (*19*), out of five clay minerals contained in bentonite, three are micaceous, one is platy crystaline, and one is amorphous. According to the hydrometer analysis 99 per cent of the bentonite particles are smaller in diameter than 0.05 millimeter (silt), 85 per cent are smaller than 0.005 millimeter (clay), 80 per cent are smaller than 0.001 millimeter (colloids), and 79 per cent are smaller than 0.0005 millimeter.

Mica flakes.—That fraction passing the No. 20 and retained on the No. 100 sieve.

Diatoms.—Celite: 95 per cent of particles smaller in diameter than 0.05 millimeter and 61 per cent smaller than 0.005 millimeter.

Peat.—Everglade peat, Florida, furnished by the United States Bureau of Chemistry and Soils. Sixty-five per cent of particles smaller in diameter than 0.05 millimeter and 18 per cent of particles smaller than 0.005 millimeter.

In addition to the tests referred to above both a compression and a slaking test may assist in identifying those binder clays of the group A–1 and A–2 subgrades which, because of certain chemical constituents, are inclined to "set up" upon drying. These same tests performed upon soil samples both treated and untreated with bituminous materials serve to disclose to some degree the increase in stability furnished by the treatment. Chemical tests may also be required to disclose those soil chemicals which are likely

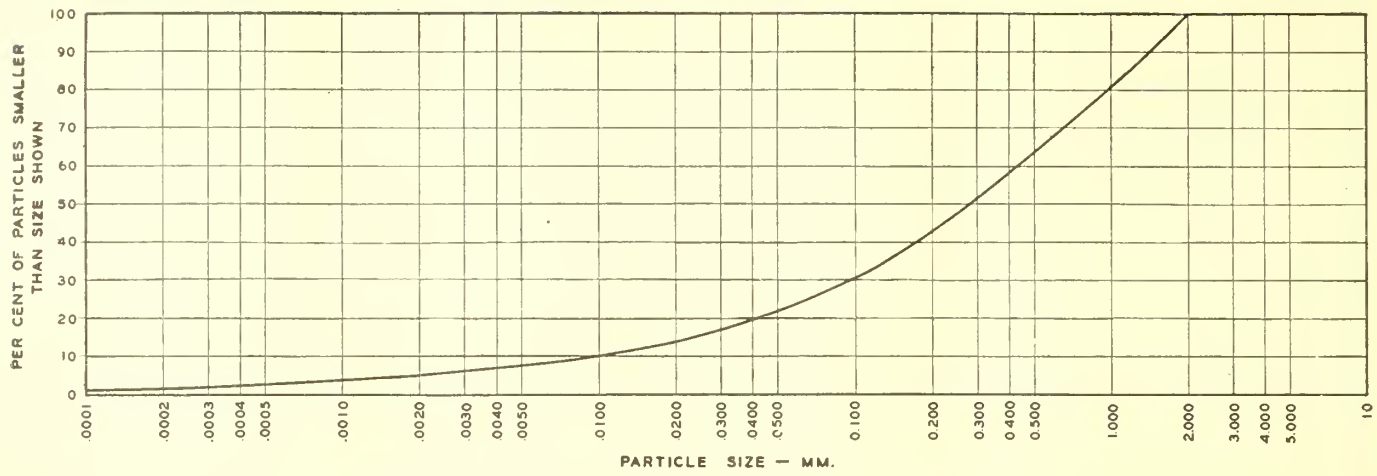


FIGURE 35.—GRAIN SIZE ACCUMULATION CURVE FOR GOOD BINDER

TABLE 5.—*Laboratory test results given by representative soil constituents*

Soil constituent	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent		Volumetric change	Lineal shrinkage
			Limit	Ratio	Centrifuge	Field		
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*	*Per cent*	*Per cent*
Sand	20	[1] 0			4	25	0	0
Silt	27	7	19	1.8	22	22	5	2
Clay	100	54	11	2.1	70	55	92	19
Colloids	399	354	6	2.0	[2]	86	160	27
Colloids, 50 per cent	174	154	12	1.9	291	54		
Mica	123	0	160	0.52	159	142	[3] —9	[3] —3
Diatoms	163	0	134	0.5	221	212	39	10
Peat	136	0	44	0.9	90	121	69	16

[1] Indicates nonplastic soils without plastic limits.
[2] Centrifuge moisture equivalent could not be determined because of very great expansion suffered by pure bentonite when being saturated.
[3] Negative value indicates expansion of mica on drying, discussed subsequently.

to exert a detrimental influence upon concrete pavements and structures. The compression, slaking, and chemical tests are not discussed in this report.

In order to use the constants intelligently one must thoroughly understand their physical significance. The soil identification chart, discussed subsequently in this report, assists in readily identifying the characteristics of many soils. The interpretation of results furnished by tests performed on many other soils, however, can be accomplished only by an intimate knowledge of physical phenomena occurring when the soils are subjected to test. For this reason an attempt is made to explain these physical phenomena in detail.

In this connection, it should be noted that the significance of a constant may vary, depending upon the degree of capillarity possessed by the soil. For this reason soils are referred to as either expansive or nonexpansive soils, according to their degree of capillarity.

The expansive soils, silt, clay, etc., are those whose capillarity is sufficient to cause swelling, shrinkage, or detrimental frost heave in appreciable amount. The nonexpansive soils are those generally termed sands, whose capillarity is not sufficient to produce these phenomena.

It is important to remember also that when a nonexpansive soil is being added to an expansive soil in increasing amounts the change in certain properties from the expansive to the nonexpansive variety may be abrupt instead of gradual. As will be shown later, sand added to clay in increasing amounts causes the resulting mixture to change its shrinkage characteristics very slowly until, when the percentage of sand becomes equal to a certain amount, the shrinkage characteristics of the mixture are abruptly changed from those of the clay to those of the sand. Important decrease in supporting value caused by increasing the moisture content of soils occurs not gradually but abruptly, when the moisture content of the soil exceeds a certain amount. In the same abrupt manner the state of the soil may change from the plastic to the semisolid or from the semisolid to the solid with small change in moisture content.

SOIL IDENTIFICATION CHART SHOWS BASIC RELATIONS

The chief value of the soil constants as a means of identifying subgrade soils lies in the relations existing between them rather than in the magnitudes of the individual constants, considered separately. The use of these interrelationships, combined with the values of the constants themselves, as a basis for the construction of the soil identification chart is a distinctive feature of the procedure developed by this bureau in its subgrade studies.

The four graphs of Figure 36 constitute the soil identification chart. They show the relations which have been found experimentally to exist between the liquid limit and four other test constants, the plastic limit, the shrinkage limit, the centrifuge moisture equivalent, and the field moisture equivalent. In the paragraphs which follow the test constants are defined, their significance is explained, and the relations which form the basis of the soil identification chart are developed.

Critical moisture.—The deformations of either confined or unconfined soil samples under constant load increase with increase of moisture content at a consistent rate until a given moisture content known as the critical moisture is reached. When the moisture content is increased above this value the deformations of the samples increase at a very much greater rate than for similar moisture increases below the critical moisture. This fact is illustrated in Figure 37, which is the reproduction of a curve published previously in PUBLIC ROADS (*12*).

It will be noted in this figure that, for a load of 5.6 pounds per square inch, the deformation increases at the rate of about 0.00026 inch for each 1 per cent increase in moisture content below 26.7 per cent, the critical moisture. For increase in moisture content above 26.7 per cent the deformations increase at the rate of 0.022 inch for 1 per cent increase in moisture content.

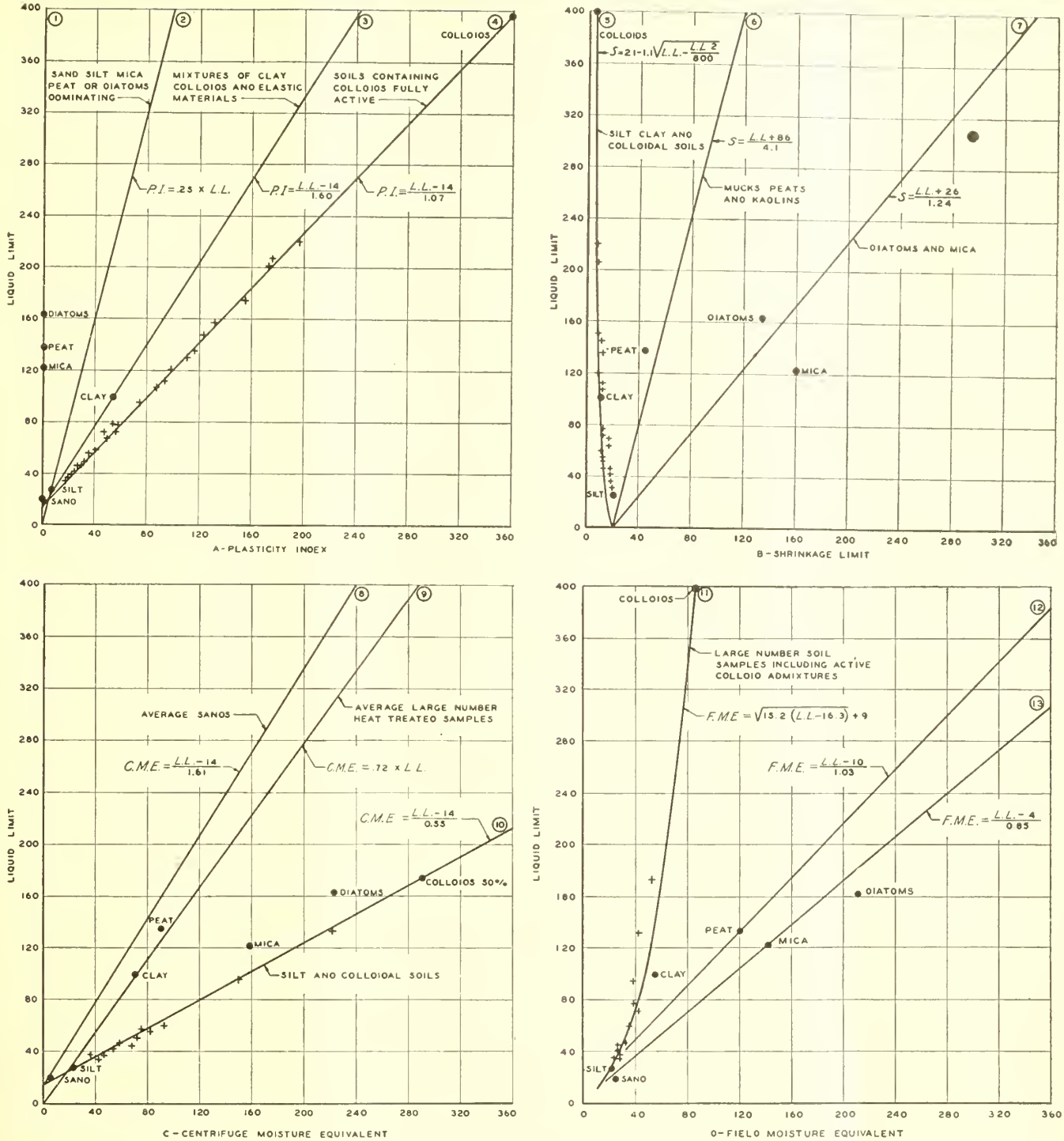


FIGURE 36.—THE SOIL IDENTIFICATION CHART: RELATIONS EXISTING BETWEEN LIQUID LIMIT AND OTHER TEST CONSTANTS

The ratio of applied load to corresponding deformations for different soils is not related to the critical moistures of these soils. As a consequence, therefore, the relation between the critical moisture of one soil and the critical moisture of another soil is not indicative of the relative supporting powers of the two soils. Furthermore, the degree of support indicated by the load-deformation relationship of soils at the critical moisture content varies widely in different soils.

The critical moisture is approximately equal to the plastic limit of cohesive soils and to 75 per cent of the liquid limit of cohesionless soils, which do not have plastic limits (*12*). The determination of the critical moisture is not a routine test. It assists in the explanation of the significance of the plasticity tests, and is of considerable practical importance, because it indicates the extent to which the moisture contents of soils must be reduced in order that they may be stabilized by drainage.

Plastic limit.—This constant is defined as the lowest moisture content, expressed as a percentage of the weight of the oven-dried soil, at which the soil can be rolled into threads one-eighth inch in diameter without the threads breaking in pieces.

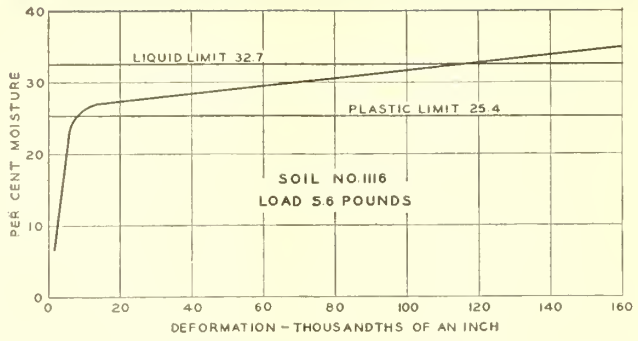


FIGURE 37.—DEFORMATION OF SOIL SPECIMEN WITH INCREASE IN MOISTURE CONTENT. NOTE SUDDEN INCREASE IN RATE OF DEFORMATION AS CRITICAL MOISTURE IS REACHED. PLASTIC LIMIT AND LIQUID LIMIT ARE ALSO MARKED ON THE CURVE

Figure 38 shows the nature of the test for determination of the plastic limit. The upper sample, having a moisture content above the plastic limit, can be rolled into threads less than one-eighth inch in diameter without crumbling under the pressure exerted by the hand. The pressure required to deform the threads varies widely with the character of the soil. The lower part of the drawing shows a soil thread which has crumbled because the moisture content of the soil has been reduced by evaporation to the plastic limit or below.

The prime importance of the plastic limit with respect to this discussion is the fact that it furnishes part of the data required for computing the plasticity index. The following significant relationships should also be noted:

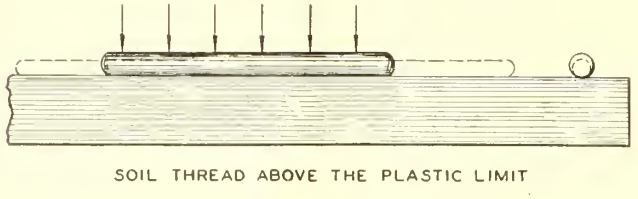


FIGURE 38.—PHENOMENON OCCURRING DURING THE PLASTIC LIMIT TEST

The plastic limit (*a*) equals approximately the moisture content at which a minimum capillary pressure of 2.5 kilograms per square centimeter acts upon the soil sample; (*b*) equals the moisture content at which the coefficient of permeability of homogeneous clays becomes practically equal to zero; (*c*) equals the moisture content above which water evaporates about 4 per cent faster from a clay sample than from the free water surface; (*d*) equals the moisture content at which the speed of evaporation starts to decrease; (*e*) equals the moisture content below which the physical properties of water are no longer identical with those of free water (*12*).

Sand, mica, diatoms, and peat have no plasticity and therefore do not have plastic limits. Silts occasionally may have plastic limits. Clay and colloids have plastic limits. Of the representative subgrade soil constituents referred to in this report (p. 119), the silt has a plastic limit equal to 20, that of the clay is 46, and that of the colloids is 45.

Liquid limit.—The liquid limit is defined as that moisture content, expressed as a percentage of the weight of the oven-dried soil, at which the soil will just begin to flow when lightly jarred 10 times. According to this definition soils at the liquid limit have a very small but definite shear resistance, which may be overcome by the application of very little force. This resistance, it should be noted, is by definition practically constant for all soils when at the liquid limit, as contrasted with the variable support furnished by different soils when at the plastic limit. At the liquid limit the cohesion is practically equal to zero (*12*).

The nature of the liquid limit test is indicated in Figure 39. The soil sample is placed in a porcelain

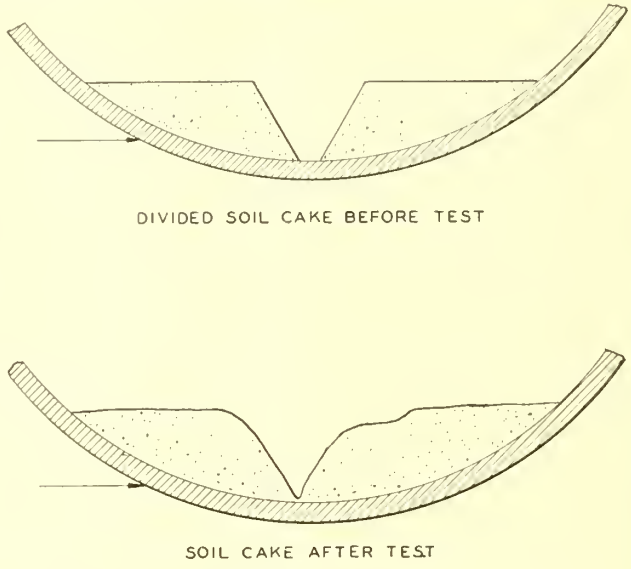


FIGURE 39.—PHENOMENON OCCURRING DURING THE LIQUID LIMIT TEST

evaporating dish about 4½ inches in diameter, shaped into a smooth layer about three-eighths inch thick at the center, and divided into two portions by means of a grooving tool. The dish is held firmly in one hand and tapped lightly 10 times against the palm of the other hand. If the lower edges of the two soil portions do not flow together as shown in the lower part of Figure 39, the moisture content is below the liquid limit. If they flow together before 10 blows have been struck, the moisture content is above the liquid limit. The test is repeated, with more or less moisture present, as the case may be, until a condition is reached where the two edges exactly meet after 10 blows have been struck. The arrows indicate the lateral flow, or shear failure, caused by the 10 blows.

The force created in the soil during the test is a function of the specific gravity of the soil particles combined with full or partial hydrostatic uplift, depending upon whether the soil is of the expansive or nonexpansive type.

Liquid limits of nonexpansive cohesionless soils indicate the moisture content required to lubricate the grain surfaces sufficiently to cause flow under the prescribed force. Liquid limits of expansive cohesionless soils indicate the degree of expansion required to reduce

to practically zero the cohesion furnished by capillary pressure, skin friction, etc. Liquid limits of expansive cohesive soils indicate the degree of expansion required to reduce to practically zero the true cohesion of the soil particles in addition to that furnished by the capillary pressure, skin friction, etc.

In this way the liquid limit serves to distinguish (*a*) sands, with respect to the resistance which they furnish

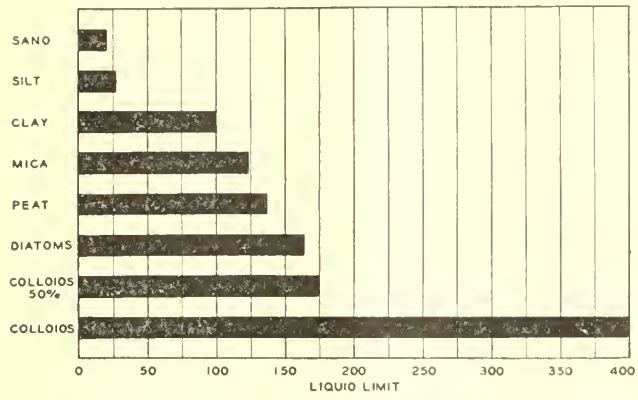


FIGURE 40.—LIQUID LIMITS OF REPRESENTATIVE SOIL CONSTITUENTS

to flowing, and (*b*) all other soils with respect to the relative volume of the pores of capillary dimension which these soils possess. The wide range in the liquid limits of different soils is illustrated by Figure 40, which show graphically the liquid limits of the representative soil constituents.

The liquid limits of the uniform subgrade groups may be stated approximately as follows:

Group A–1.—Generally greater than 14 and less than 25.
Group A–2.—Generally greater than 14 and less than 35.
Group A–3.—Varies in value from slightly less than 10 to slightly more than 35. Small liquid limits, such as 10 to 14, signify the beach and other rounded sand grains which, when sufficiently wetted, will slide over each other, i. e., flow, because of the lubrication of their surfaces. Grains uniform in size and perfectly spherical in form would probably flow without being lubricated. An abnormally large liquid limit, 30 to 35 in the case of sands, signifies high resistance to sliding furnished by a high degree of surface roughness or angularity of grain.
Group A–4. Generally greater than 20 and less than 40.
Groups A–5, A–6, and A–7.—Usually greater than 35.
Group A–8.—Likely to be greater than 45.

Plasticity index.—This term is defined as the difference between the liquid limit and the plastic limit. Plasticity indices equal to zero designate nonplastic soils, i. e., those which have no plastic limits.[8] At the plastic limit the soil particles may be considered as having acquired a degree of lubrication sufficiently high to permit them to slide over each other when loaded, although still possessing cohesion in appreciable amount. At the liquid limit, according to definition, the soil particles are separated to such an extent that practically no cohesion exists between them.

It follows, therefore, that the difference between the plastic limit and the liquid limit indicates the increase in moisture content required to increase the thickness of the water films separating the soil particles to a degree such that the cohesion existing between them is reduced practically to zero. Thus the plasticity index may be considered as a measure of the cohesion possessed by the soil.

[8] Only a very few soils of the thousands tested in the subgrade laboratory of the Bureau of Public Roads have had plastic limits equal to the liquid limits.

As stated above, this cohesion consists of two parts: (*a*) That furnished by capillary pressure, skin friction, etc., and (*b*) that furnished by the true cohesion (molecular attraction) of the soil particles. To illustrate, assume that a pebble is first immersed and then removed from water. The adhesion existing between the surface of the pebble and the water particles in intimate contact with it is very high, but decreases rapidly in amount as the distance separating the water molecule from the pebble surface increases. Consequently the water molecules separated farthest from the pebble surface flow off under the force of gravity.

Two soil particles may be held together by both the adhesion possessed by water for the surface of the soil particles and the cohesion existing between water molecules. With increasing thickness of water film separating the two soil particles the attraction which tends to hold them together rapidly decreases to the vanishing point.

True cohesion existing between soil particles, like the adhesion existing between water molecules and the pebble surface, decreases rapidly in degree as the distance separating the soil particles increases. When the two soil particles possess true cohesion they are held together by a force exceeding that furnished by the molecular attraction of water. Consequently, the thickness of water film required to overcome the true cohesion existing between soil particles is greater than that required to overcome the adhesion due to capillary pressure, skin friction, etc.

If two glass plates are wetted with a small amount of water and pressed together they can not be pulled apart without the application of an appreciable external force. A very slight force, however, may serve to cause one plate to slide upon the other. This condition is analogous to the moisture content of the soil when at the plastic limit. By increasing the thickness of water film separating the two plates, one may be caused by only its own weight to separate from the other. This is analogous to the moisture content of the soil when at the liquid limit. The increase in thickness of water film required to produce the change from the sliding to the separated state represents that portion of the plasticity index required to equalize only the cohesion furnished by the molecular attraction existing between the water and the surfaces of the glass plates.

If, before being wetted and pressed together, the glass plates had been coated with a gluey colloid, the thickness of water film required to produce the change from the sliding to the separated state would of necessity have been larger than if the plates had not been so coated. The difference between the amount of water required to change the coated and the uncoated plates from the sliding to the separated state may be regarded as analogous to the amount of water required to overcome the cohesion possessed by the colloidal particles.

It was stated above that the critical moisture of cohesionless soils equals 75 per cent of the liquid limit. It follows that 25 per cent of the liquid limit may be assumed to indicate the amount of water required to overcome the cohesion furnished by the capillary pressure, skin friction, etc., in soils. If the same principle is applied to cohesive soils, it may be assumed that that portion of the plasticity index exceeding 25 per cent of the lower liquid limit is the amount of water required to overcome the true cohesion existing between the individual soil particles. Consequently, the quantity obtained by subtracting 25 per cent of the liquid limit

from the plasticity index, indicates the relative amounts of true cohesion possessed by the surfaces of the soil particles. Thus the relation existing between the liquid limit and the plasticity index may serve very well to disclose certain characteristics of the soil.

Figure 36, A, illustrates relations existing between the plasticity index and the liquid limit. The large dots denote the relations given in Table 5 for representative soil constituents. Curve 1, indicating a plasticity index of zero, is characteristic of sand, mica, peat, and diatoms. Curve 2 represents the relation, plasticity index $= 0.25 \times$ liquid limit. This relation is characteristic of soils containing sand, silt, mica, peat, or diatoms in dominating amounts. Curve 3 indicates the statistical relationship given by tests of a very large number of samples of soils containing clay, colloids, and elastic materials such as peat or mica, and also soils containing colloids not fully active because of either heat treatment or state of flocculation. Curve 4 shows the relation given by compressible mixtures of fully active colloids. The crosses adjacent to curve 4

FIGURE 41.—SHRINKAGE OF REPRESENTATIVE SOIL CONSTITUENTS. TOP ROW, LEFT TO RIGHT, DIATOMS, PEAT, COLLOIDS; BOTTOM ROW, LEFT TO RIGHT, MICA, CLAY, SILT, SAND

denote results given by tests of soil samples containing admixtures of bentonite, 1.5 to 50 per cent.

The members of the uniform subgrade groups generally have plasticity indices as follows:

Group A–1.—Usually less than 8 and generally related to their liquid limits according to the relation shown by curve 2.

Group A–2.—Usually less than 15.

Group A–3.—Have no plasticity; therefore, do not have plasticity indices.

Group A–4.—Less than those indicated by curve 3.

Group A–5.—Seldom greater than those indicated by curve 3.

Group A–6.—Approximately equal to those indicated by curve 4.

Group A–7.—Generally greater than those indicated by curve 3 and smaller than those indicated by curve 4.

Group A–8.—Generally less than those indicated by curve 3.

Shrinkage limit and shrinkage ratio.—The mechanics of soil shrinkage was described in Part I of this report (PUBLIC ROADS, June, 1931, pp. 102 and 103) and will not be repeated here. The shrinkage limit is defined as the moisture content, expressed as a percentage of the dry weight, required to fill the pores of a soil sample which has been dried to constant weight from a moisture content sufficient in amount to fill the soil pores completely. Those soils which shrink during this drying process are referred to as possessing significant shrinkage limits. The shrinkage limits of nonexpansive soils such as sand, etc., which do not shrink during this drying process may be computed, as shown later, when the specific gravities of the soils are known. Such shrinkage limits are termed "theoretical" to distinguish them from the "significant" shrinkage limits. Sand and mica have theoretical and the other constituents have significant shrinkage limits. In general, only the significant limits are given in routine test reports.

Figure 41 illustrates the shrinkage of representative soil constituents. The pats shown were dried to constant weight from a moisture content slightly above the liquid limit. The container in which the pat rests represents the original volume of the pat.

The capillary pressure per square centimeter exerted upon the surface of the drying soil sample may be computed as follows:

Assume the voids in the soil mass to be of square cross section and to have each a width equal to a. The perimeter of each tube equals $4a$. Since the force exerted by capillary pressure is equal to 0.0764 grams per centimeter (*1*), the force exerted upon each tube, is given by the expression, $4\ a \times 0.0764 = 0.306a$.

If we should assume that the soil surface is completely covered by such openings, the number of openings per square centimeter equals the reciprocal of the area, a^2, of each opening. Consequently, when we designate *S. F.* as the force causing shrinkage in a soil, we have

$$S.\ F. = \frac{0.306a}{a^2} = \frac{0.306}{a}. \qquad (14)$$

According to this relation, illustrated graphically in Figure 42, the pressure exerted upon a soil possessing voids 0.1 millimeter wide equals 30.6 grams per square centimeter and that exerted upon a soil possessing voids 0.001 millimeter wide equals 3,060 grams per square centimeter. This is the explanation of the fact, illustrated in Figures 41 and 43, that colloids upon drying compact the greatest amount, clay a less, and silt a still less amount.

The shrinkage ratio is defined as the volume change, expressed as a percentage of the volume of the dry soil cake, divided by the moisture loss above the shrinkage limit, expressed as a percentage of the weight of the dry cake.

A determination of skrinkage limit and shrinkage ratio is illustrated in Figure 44. As a soil pat consisting of Yaguajay clay was being dried from the wet state, the changes of both volume and moisture content were determined at different times. The results of these observations are plotted as small circles in the figure. The abscissa of any point on the curve denotes volume change expressed as a percentage of the volume of the dry pat; the ordinate denotes moisture content, expressed as a percentage of the weight of the dry pat.

These points, it will be noted, lie practically on a straight line which intersects the zero line at a moisture content of 11.1 per cent. Consequently no volume change will occurr in the soil cake when the moisture content is reduced from 11.1 to 0 per cent, and 11.1 per cent is the shrinkage limit.

As the moisture content was reduced from 107.8 to 11.1 per cent, the pat underwent a volume change equal to 200 per cent of its volume when dry. The shrinkage ratio is therefore obtained by the computation,

$$\frac{200}{107.8-11.1} = \frac{200}{96.7} = 2.07$$

The shrinkage limit, the shrinkage ratio, and the specific gravity are interrelated as follows:

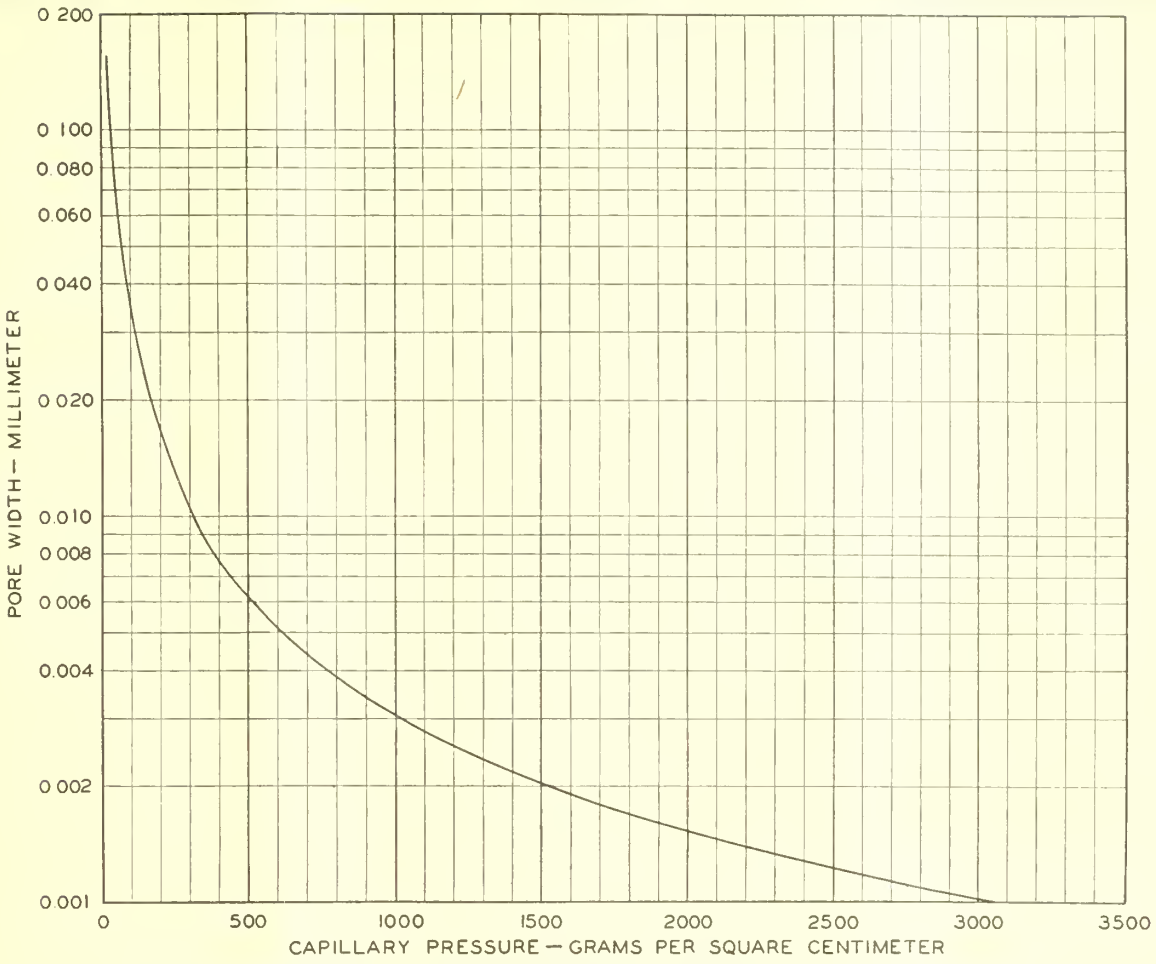


FIGURE 42.—THEORETICAL RELATION BETWEEN PORE WIDTH AND CAPILLARY PRESSURE

Let

S = shrinkage limit.
w = moisture content.
V = volume of wet soil cake.
V_o = volume of dry soil cake.
W = weight of wet soil cake.
W_o = weight of dry soil cake.
R = shrinkage ratio.
G = specific gravity of soil particles.

The total volume change, $V-V_o$, is equal to the moisture loss occurring between the original moisture content, w, and the shrinkage limit, S.

The weight of water lost between these two limits is given by the expression,

$$\frac{w\times W_o}{100}-\frac{SW_o}{100}=\frac{W_o\,(w-S)}{100}$$

If measurements are made in grams and centimeters,

$$\frac{W_o\,(w-S)}{100}=\text{volume of water lost.}$$

$$=V-V_o$$

$$W_o w-W_o S=(V-V_o)\times 100$$

$$W_o S=W_o w-(V-V_o)\times 100$$

$$S=w-\frac{V-V_o}{W_o}\times 100 \qquad (15)$$

And

$$w-S=\frac{V-V_o}{W_o}\times 100$$

The shrinkage ratio, R, defined as the volume change in percentage of the dry volume, divided by the moisture loss above the shrinkage limit in percentage of the dry weight, is given by the equation,

$$R=\frac{\frac{V-V_o}{V_o}\times 100}{w-S}$$

Substituting the value of $w-S$ previously obtained, we have

$$R=\frac{\frac{V-V_o}{V_o}\times 100}{\frac{V-V_o}{W_o}\times 100}=\frac{W_o}{V_o} \qquad (16)$$

The specific gravity equals the weight of the dry soil in grams divided by its true volume in cubic centimeters. The true volume of the dry soil equals the apparent volume, V_o, minus the water content at the shrinkage limit, $\frac{SW_o}{100}$. We have, therefore,

$$G=\frac{W_o}{V_o-\frac{SW_o}{100}}=\frac{1}{\frac{V_o}{W_o}-\frac{S}{100}}=\frac{1}{\frac{1}{R}-\frac{S}{100}} \qquad (17)$$

At one determination of the volume of the Cuban (Yaguajay) soil the weight of the wet pat, W, was 43.74 grams and its volume, V, was 28.04 cubic centimeters. In the dried state the weight of the pat, W_o, was 25.08

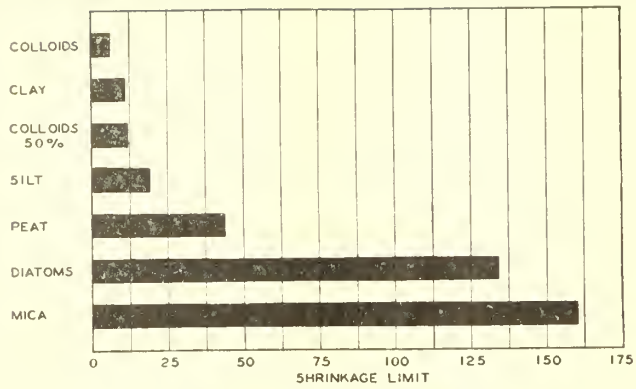


FIGURE 43.—SHRINKAGE LIMITS OF REPRESENTATIVE SOIL CONSTITUENTS

grams and its volume, V_o, was 12.16 cubic centimeters. The value of the moisture content, w, of the wet soil is given by the expression

$$w = \frac{W - W_o}{W_o} \times 100 = \frac{43.74 - 25.08}{25.08} \times 100 = 74.40 \text{ per cent.}$$

And the shrinkage limit,

$$S = w - \frac{V - V_o}{W_o} \times 100 = 74.40 - \frac{28.04 - 12.16}{25.08} \times 100 =$$

11.1 per cent.

This value checks with that shown in Figure 44.

The volume change which occurred during the loss of 74.4 per cent of moisture, is given by the equation

$$C_o = \frac{V - V_o}{V_o} \times 100 = \frac{28.04 - 12.16}{12.16} \times 100 = 130.6 \text{ per cent.}$$

This value is shown in Figure 44 by means of a double circle.

The shrinkage ratio $R = \frac{W}{V_o} = \frac{25.08}{12.16} = 2.06$ as compared with 2.07, the value obtained from Figure 44.

It will be noted that the shrinkage ratio R is also the apparent specific gravity of the soil. Thus the weight of a cubic foot of the Cuban clay in the dried state equals $62.5 \times 2.06 = 128.75$ pounds.

The specific gravity computation may be subject to appreciable error, since it is based on the assumption that the relation which exists between moisture loss and percentage volume change is as consistent as that indicated in Figure 44. Certain elastic soils are likely to expand during the period when the moisture content is being reduced from the shrinkage limit to zero. Mica is one of these materials and the degree to which it may expand under these conditions is illustrated in Figure 45. This accounts in part for the high shrinkage limit given for mica in Figure 43. Furthermore, the presence of air in the pores of a drying soil may prevent it from shrinking exactly in accordance with the relation shown in Figure 44. Figure 46 shows how different soils may vary in drying from the relation as stated. Table 6 illustrates the variation which may exist between actual and computed specific gravities of soils.

TABLE 6.—*Comparison of specific gravities as obtained by actual determination and by computation from formula 17*

Soil No.	Specific gravity, actual determination	Specific gravity computed from formula	Soil No.	Specific gravity, actual determination	Specific gravity computed from formula
2,262	2.72	2.70	2,242	2.640	2.67
2,264	2.70	2.75	2,499	2.815	2.80
2,280	2.59	2.55	2,443	2.752	2.77
2,235	2.50	2.38	1,688	2.716	2.72
2,278	2.64	2.58	1,606	2.740	2.75
2,261	2.62	2.72	1,547	2.653	2.67
2,272	2.65	2.71	1,611	2.612	2.62
2,243	2.71	2.79	2,370	2.715	2.72
2,251	2.54	2.62	2,368	2.660	2.66
2,248	2.63	2.65	2,365	2.640	2.51
2,250	2.71	2.67			

Figure 36, B illustrates relations existing between the shrinkage limit and the liquid limit. The large dots denote the relations given in Table 5 for the representative soil constituents. Curve 5 is characteristic

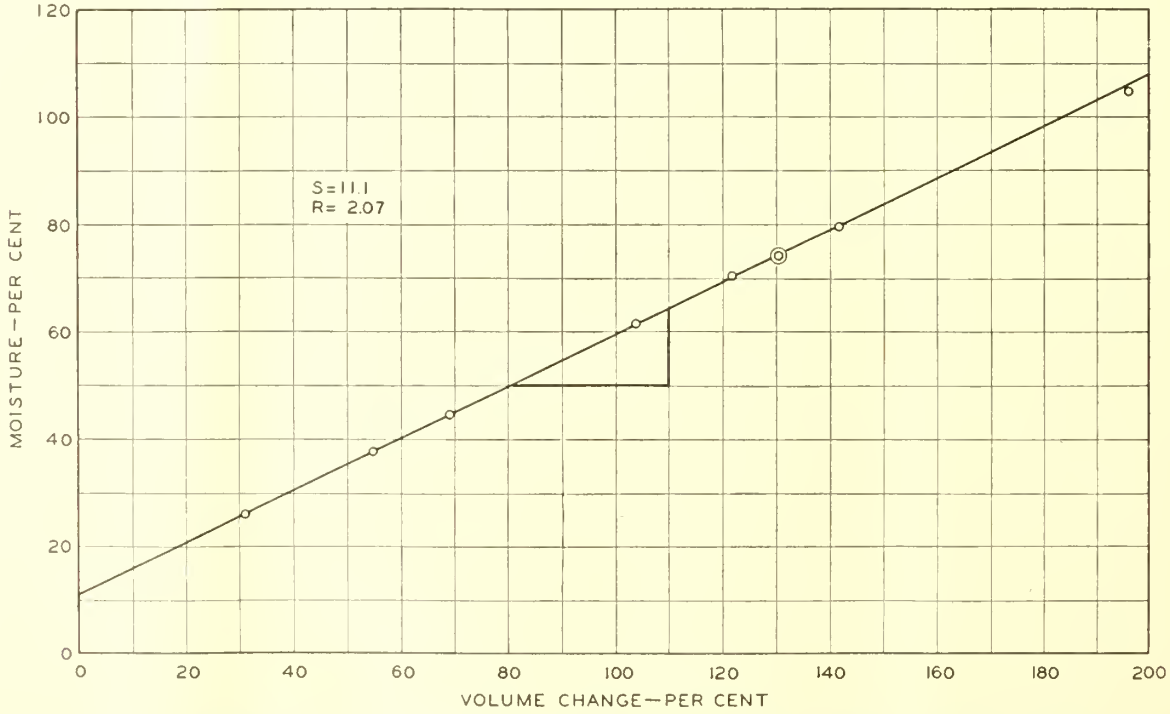


FIGURE 44.—RELATION BETWEEN MOISTURE CONTENT AND VOLUME CHANGE FOR YAGUAJAY CLAY

FIGURE 45.—SIDE VIEW OF MICA PAT SHOWING EXPANSION ON DRYING TO CONSTANT WEIGHT FROM A MOISTURE CONTENT SLIGHTLY ABOVE THE LIQUID LIMIT. THE MICA JUST FILLED THE CUP WHEN WET

of silt, clay, and colloidal soils. It also indicates approximately the average relation given by tests of a large number of natural soil samples (*21*). The small crosses adjacent to this curve represent results of tests performed on natural soil containing bentonite in admixtures varying between 1.5 and 50 per cent. Curve 6 indicates the moderately high shrinkage limits characteristic of mucks, peats, and kaolins. Curve 7 represents diatoms and mica, which have high shrinkage limits.

Shrinkage limits characteristic of the uniform subgrade groups are as follows:

Group A–1.—Generally greater than 14 and less than 20.

Group A–2.—May be either theoretical or significant depending upon other constants. Not likely to exceed 25 when significant.

Group A–3.—No significant shrinkage limit.

Group A–4.—Generally less than 25. Increase in expansive properties of members of this group indicated when shrinkage limits exceed 20 and approaches relationship represented by curve 6.

Group A–5.—Generally greater than 30 and greater than 50 for very undesirable members of this group. May approach relation indicated by curve 6 for silts containing peat and approach relation indicated by curve 7 for soils containing either diatoms or mica in appreciable amount.

Group A–6.—Not likely to exceed in appreciable amount values represented by curve 5.

Group A–7.—For flocculated varieties of this group, may slightly exceed values given by curve 5. For varieties containing mica, peat or diatoms, may exceed very appreciably values indicated by curve 5, but generally less than those indicated by curve 6. Soils of this group subject to frost heave have the higher shrinkage limits.

Group A–8.—Generally in neighborhood of values indicated by curve 6, and seldom greatly in excess of those values.

Centrifuge moisture equivalent.—This constant is defined as the moisture content, expressed as a percentage of the weight of the oven-dried soil, retained by a soil sample after first being soaked in water for 6 hours, then drained in a humidifier for 12 hours, and finally centrifuged under an acceleration of 1000 × gravity for 1 hour.

The action which takes place is illustrated in Figure 47. Water is forced outward through the bottom of the cup under the influence of two forces, the centrifugal force acting on the water and the pressure which the soil particles exert on one another. The centrifugal force acting on the water is proportional to the distance from the surface of the sample. Since the acceleration is $1000g$ the pressure at a distance y will be equal to y kilograms per square centimeter. The pressure which the individual particles exert upon each other in the direction of the axis of the cup is a function of the specific gravity of the particles, the distance from the surface, and the extent of hydrostatic uplift. If the

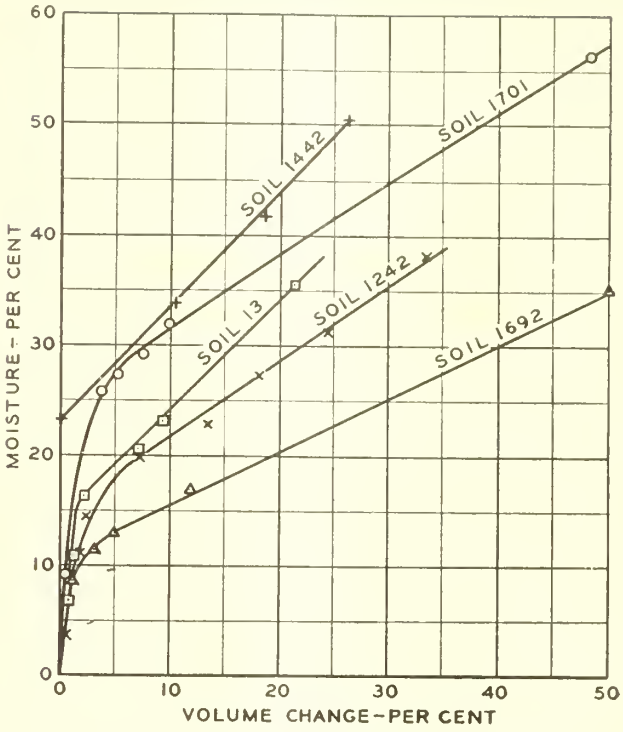


FIGURE 46.—TYPICAL CURVES SHOWING VARIATION OF VOLUME CHANGE WITH MOISTURE CONTENT

soil particles are not surrounded by water, the hydrostatic uplift is negligible, and we may define the pressure on the particles as Py kilograms per square centimeter, where P is a function of the specific gravity. This is the case with nonexpansive soils. In the case of expansive soils the hydrostatic uplift is much greater. If we assume it as full, the pressure becomes $(P–1)y$, or, at the bottom of the cup, $(P–1)h$.

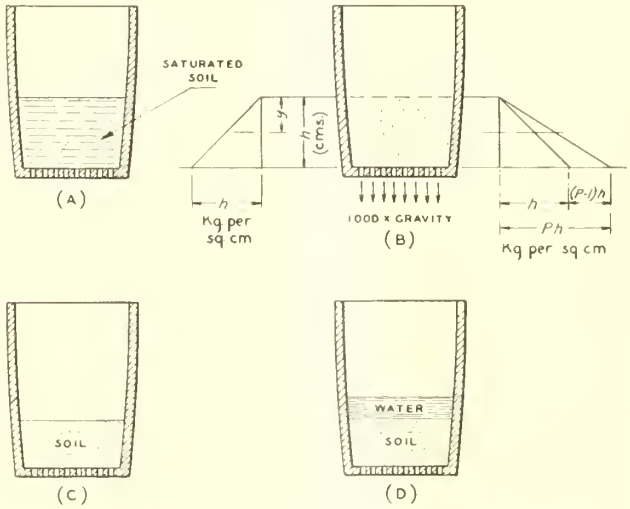


FIGURE 47.—PHENOMENON OCCURRING DURING THE CENTRIFUGE MOISTURE EQUIVALENT TEST

This combination of forces results in pressures which are equivalent to an average pressure on the sample of about 2 kilograms per square centimeter. As the intimacy of contact between soil particles is increased, the tendency is for water to be squeezed from the sample at both top and bottom. The tendency for water to be forced to the top is resisted by the centrifugal force acting on the water. This force is in turn opposed by

the frictional resistance to flow offered by the surfaces of the soil pores and the capillarity of the soil, both of which increase as the soil mass decreases in volume during the test. If the resisting force is greater than the centrifugal force water remains at the top of the sample, producing the condition called waterlogging. The amount of water which is forced through the sample and escapes through the outlets at the bottom of the cup is thus dependent on the permeability of the soil.

Thus the centrifuge moisture equivalent (*a*) serves to distinguish soils which are permeable (sand, silt, mica, diatoms, peat or flocculated clay dominating) from those which are impermeable (clay and colloids dominating) when compressed by a centrifugal force equal to about 2 kilograms per square centimeter (*12*); (*b*) serves to disclose to some extent the degree of capillarity possessed by permeable soils; (*c*) furnishes a means of distinguishing permeable soils of the non-expansive from those of the expansive varieties.

The moisture equivalents of permeable soils, for instance, decrease consistently when the sand content of the soils is increased. The shrinkage limits of silt and clay soils having capillarity in appreciable amount increase at a very slow rate with increase in the sand content of the soils until the amount of sand added be-

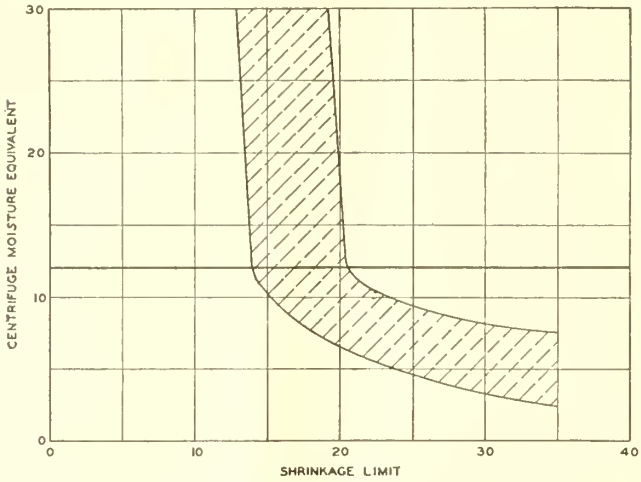


FIGURE 48.—RELATION BETWEEN CENTRIFUGE MOISTURE EQUIVALENT AND SHRINKAGE LIMIT FOR SAND ADMIXTURES

comes sufficient to reduce the capillary properties of the soil very appreciably. At this sand content the shrinkage limits of the soils become theoretical instead of significant and increase at an abnormally high rate with further additions of sand. Therefore, by plotting the centrifuge moisture equivalents against the corresponding shrinkage limits of soils to which sand has been added in increasing amounts, the centrifuge moisture equivalent value at which the shrinkage limits suddenly begin to increase is easily determined. This value of the centrifuge moisture equivalent should indicate the degree of capillarity below which expansion and shrinkage become negligible in amount.

Figure 48 shows the results furnished by a determination of this character. The soil constants were obtained from tests performed upon a number of soils containing sand in amounts varying between 20 and 80 per cent. The results, it will be noted, are grouped in a well defined band which indicates a very pronounced increase in the theoretical shrinkage limits when the amount of contained sand is sufficient to reduce the centrifuge moisture equivalents to less than 12.

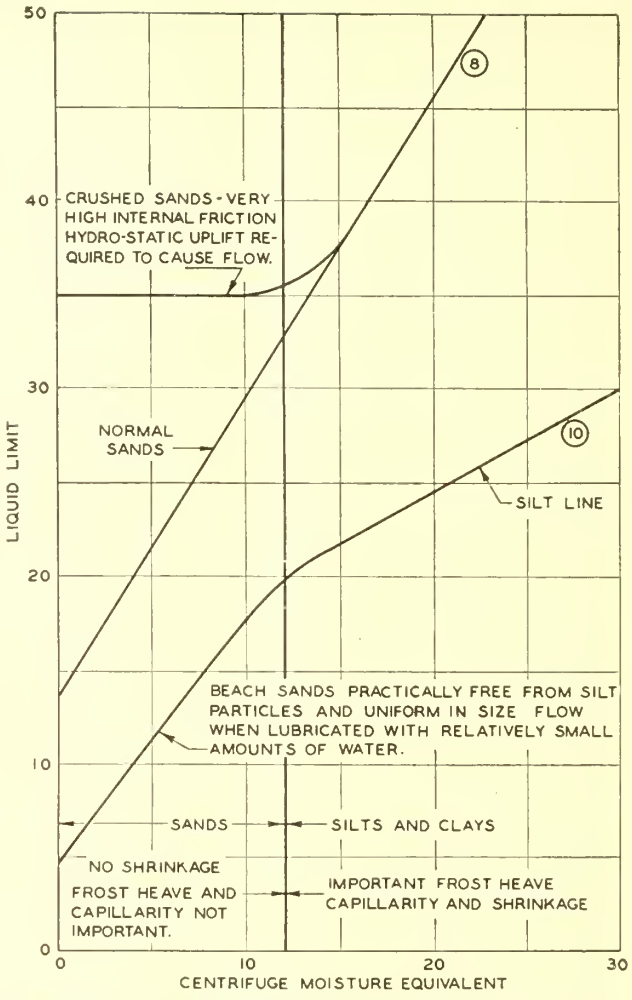


FIGURE 49.—RELATION BETWEEN LIQUID LIMIT AND CENTRIFUGE MOISTURE EQUIVALENT FOR DIFFERENT TYPES OF SAND

Since detrimental frost heave occurs only in soils having capillarity in appreciable amount, it should not occur in soils which have centrifuge moisture equivalents less than about 12. And this seems to be verified by the subgrade surveys of the bureau now in progress. To provide a proper factor of safety, however, materials used for porous (*7*) base courses in practice, should have centrifuge moisture equivalents not exceeding 6 or 8.

The centrifuge moisture equivalents of the representative soil constituents (Table 5) are shown graphically in Figure 36, C. This figure contains also curves 8, 9, and 10, which indicate important relations existing between the centrifuge moisture equivalent and the liquid limit. Curve 8 represents the relation between the liquid limits and the centrifuge moisture equivalents of average sands (absence of hydrostatic uplift). Curve 9 represents the statistical relation between the average liquid limits and centrifuge moisture equivalents obtained from a great number of heat-treated soil samples (*21*). Curve 10 represents the relation between the liquid limits and the centrifuge moisture equivalents of average silt and colloidal soils (particles acting under full hydrostatic uplift).

Members of the uniform subgrade groups may have centrifuge moisture equivalents as follows;

Group A–1.—Seldom appreciably greater than 15.
Group A–2.—Not likely to exceed 25.

Group A–3.—Not likely to exceed 12. In combination with the liquid limit, discloses the relative resistance to flowing possessed by sands equal in degree of capillarity. (See Fig. 49).

Group A–4.—Generally greater than 12, approaching values indicated by curve 10, but not likely to waterlog, although exceptions occur. When greater than the liquid limits in the absence of waterlogging, indicates especially unstable silts.

Group A–5.—Greater than 12 and not likely to waterlog, although exceptions occur. Often has values between curves 9 and 10. May approach those indicated by curve 8 for sand-mica mixtures.

Group A–6.—May approach values indicated by curve 10 for the highly colloidal soils, with values lying between curves 9 and 10 for clay soils containing sand in appreciable amount. Likely to waterlog when exceeding 40.

Group A–7.—Generally greater than values indicated by curve 9 and less than values indicated by curve 10. May not waterlog with centrifuge moisture equivalents as high as 90.

Group A–8.—Generally between curves 9 and 10.

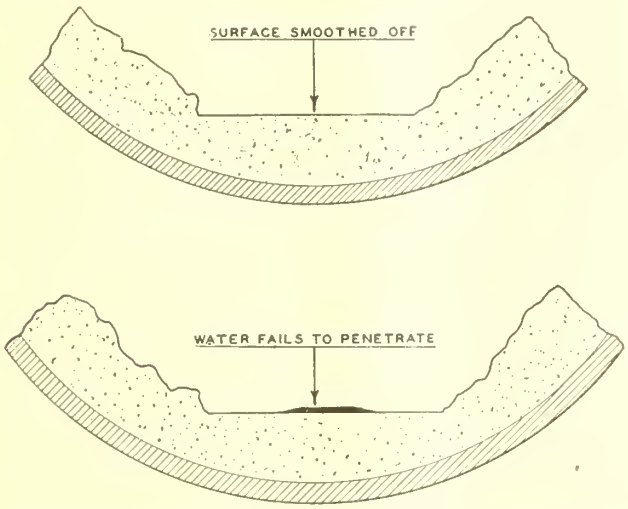


FIGURE 50.—PHENOMENON OCCURRING DURING THE FIELD MOISTURE EQUIVALENT TEST

It is interesting to note also that adding sand to the silt and clay mixes, which are represented by curve 10, will change these soils to graded mixes of the Group A–1 or Group A–2 variety and the test relationship will approach that indicated by curve 9. Adding both sand and mica to a silt may cause its centrifuge moisture equivalent to assume the relation to the liquid limit represented by curve 9.

Field moisture equivalent.—This term is defined as the minimum moisture content, expressed as a percentage of the weight of the oven-dried soil, at which a drop of water placed on a smoothed surface of the soil will not immediately be absorbed, but will instead spread out over the surface and give it a shiny appearance. (See fig. 50.)

The drop of water fails to penetrate the wet and smoothed soil sample (*a*) when the pores of nonexpansive soils (sands) are completely filled, (*b*) when the capillarity of cohesionless expansive soils (diatoms and mica) is completely satisfied, and (*c*) when cohesive soils possess moisture in amount sufficient to cause the smoothed surface of the sample to become impervious. This impervious skin may occur at moisture contents far below those required to satisfy the capillarity of cohesive soils.

That the moisture content at which the impervious skin is formed measures a definite soil property and is not dependent upon the time during which the soil remains wetted is evidenced by the fact that highly colloidal clays, whether wetted for several minutes or for 24 hours, generally resist the penetration of the drop of water at practically equal moisture contents.

Figure 36, D contains curves showing relations existing between the liquid limit and the field moisture equivalent. Curve 11 represents the statistical relationship which was found to exist between the averages of results furnished by tests performed upon a large number of natural soil samples (*21*). This curve, it will be noted, represents also the relation given by results of tests performed upon the soils containing admixtures of active colloids.

The positions of curves 12 and 13 were chosen arbitrarily to represent high and very high field moisture equivalents. The field moisture equivalents of the representative soil constituents (Table 5) are also shown graphically in Figure 36, D. The field moisture equivalent individually and in its relation to the other constants serves to furnish the following supplementary information with respect to the identification of subgrade soils.

Group A–1.—Field moisture equivalent not significant.

Group A–2.—Field moisture equivalent not significant.

Group A–3.—Field moisture equivalent indicates the porosity of these cohesionless materials when completely saturated; in combination with the liquid limit discloses the degree of saturation required to cause sands to have a very small shear resistance.

Group A–4.—When approximately equal to or larger than centrifuge moisture equivalents the field moisture equivalents indicate presence of expansive properties in detrimental amounts.

Group A–5.—Field moisture equivalents may approach values indicated by curve 12 for silts containing peat in appreciable amount and those indicated by curve 13 for highly elastic silts containing mica or diatoms in appreciable amount. May not exceed those indicated by curve 11 for kaolins possessing good binder properties.

Group A–6.—May approach values indicated by curve 11 generally, but smaller when the grading of the colloidal clay soils of this group is such as to cause smoothed surface of soil when wetted to become highly impermeable.

Group A–7.—Soils of this group either flocculated or containing organic matter partially decomposed into the colloidal state

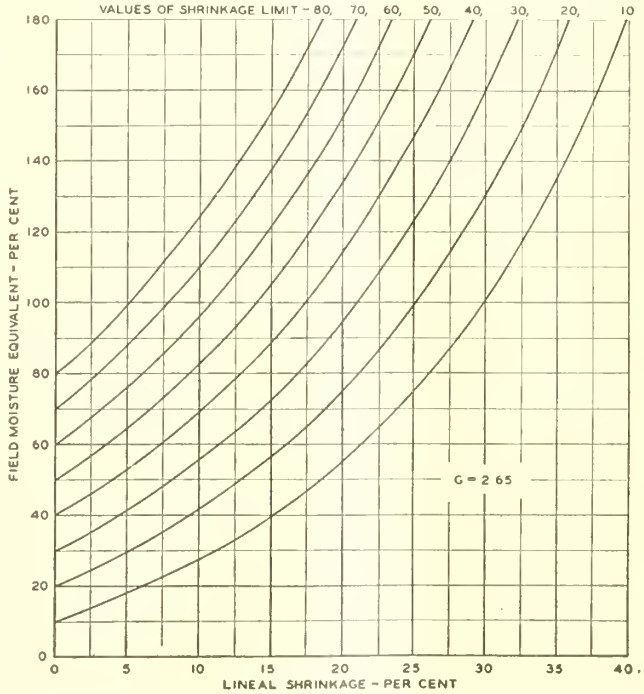


FIGURE 51.—CHART FOR ESTIMATING SHRINKAGE LIMIT FROM VALUES OF FIELD MOISTURE EQUIVALENT AND LINEAL SHRINKAGE

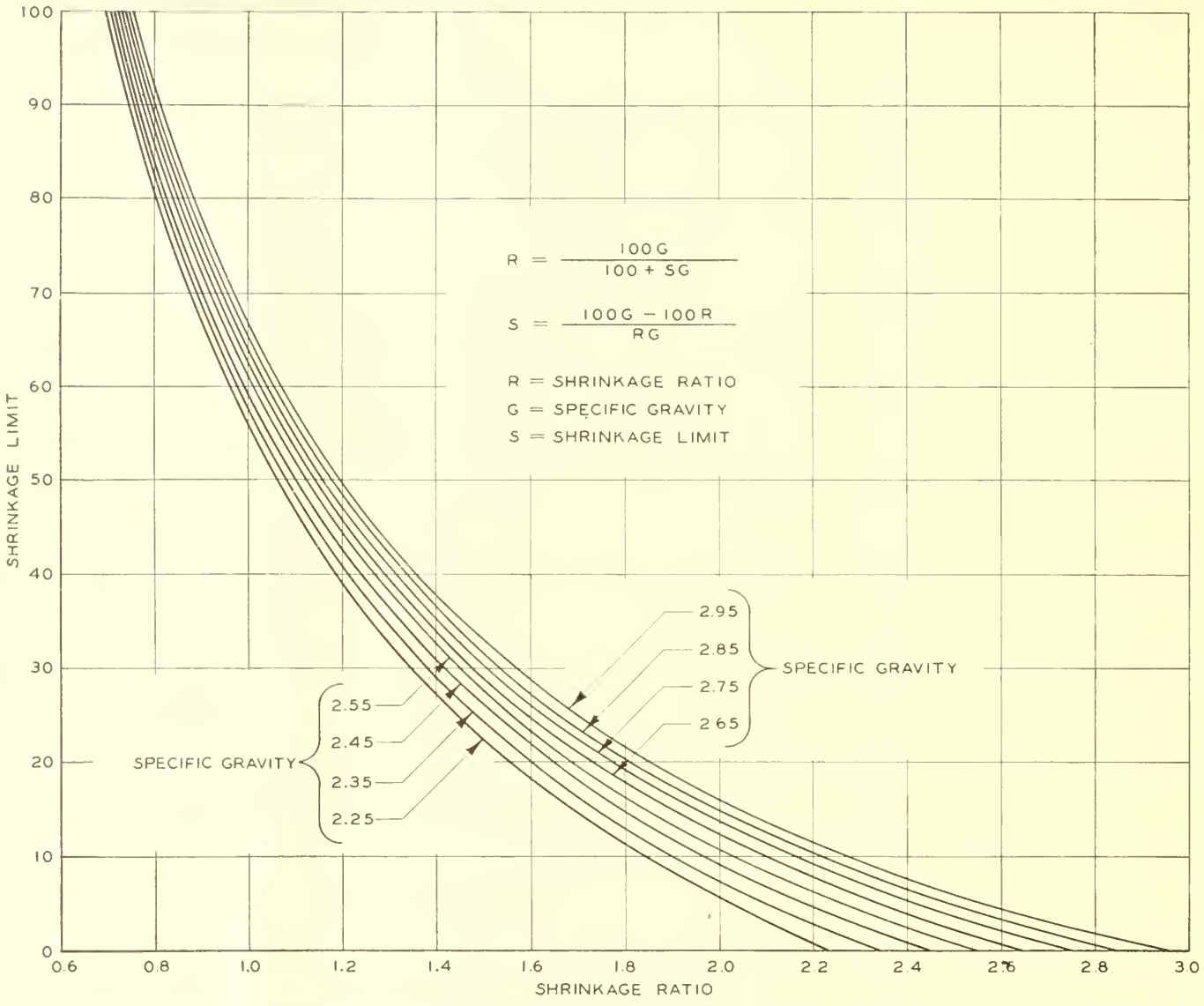


FIGURE 52.—RELATION BETWEEN SHRINKAGE LIMIT AND SHRINKAGE RATIO FOR DIFFERENT VALUES OF SPECIFIC GRAVITY

may have, as in the case of the Group A–6 soils, shrinkage limits approaching those represented by curve 5. Their field moisture equivalents, however, are likely to be appreciably greater than those indicated by curve 11.

Group A–8.—Same as group A–5.

Volumetric change.—Only in certain instances does the degree to which soils may shrink when dried out from an arbitrary wet state furnish information with respect to the identification of subgrade soils supplementary to that furnished by the constants discussed above. Volumetric changes from the field moisture equivalent are now computed only to determine whether those of graded materials and silty clays are larger or smaller than about 17. This limit, according to computation, is equivalent to a lineal shrinkage of 5, a value which has been established by A. C. Rose and C. H. McKesson (PUBLIC ROADS, August, 1924, September, 1925, and September, 1927), as representing the maximum degree of shrinkage properties which may be possessed by good soil mortars or stable subgrade soils. Thus, in certain instances the volumetric change assists in distinguishing the Group A–6 and Group A–7 soils which are inclined to shrink in appreciable amount (volumetric change approximately equal to or greater than 17) from the Group A–2, A–4, or A–5 varieties in which shrinkage is not important.

Let

$F.\ M.\ E.$ = field moisture equivalent,
G = specific gravity of soil particles,
S = shrinkage limit,
R = shrinkage ratio.

The volumetric change, C_f, is given by the formula

$$C_f = (F.\ M.\ E. - S) \times R \qquad (18)$$

We have

$$G = \frac{1}{\frac{1}{R} - \frac{S}{100}} \qquad (17)$$

Hence,

$$R = \frac{100G}{100 + GS} = \frac{1}{\frac{1}{G} + \frac{S}{100}};$$

And, by substitution,

$$C_f = \frac{F.\ M.\ E. - S}{\frac{1}{G} + \frac{S}{100}} \qquad (19)$$

No computation of the volumetric change is required for the average clay soils of the Group A–6 or Group A–7 variety when the shrinkage limits do not exceed those shown by curve 5 and the field moisture equivalents are not less than those indicated by curve 11. With a liquid limit of 35, for instance, such soils have shrinkage limits not exceeding 15 and field moisture equivalents not less than 26. By substitution of these values in equation 18, we find that the volumetric change, with an average R equal to 1.8, exceeds 20, and for this value the lineal shrinkage is larger than 5.

Lineal shrinkage.—The volumetric change, C_f, is expressed as a percentage of the dry volume of the soil cake. The lineal shrinkage, *L. S.*, expressed as a percentage of the length of wet soil bar, is defined by the formula

$$L.\ S. = 100\left[1 - \sqrt[3]{\frac{100}{C_f + 100}}\right] \qquad (20)$$

The lineal shrinkage, as such, possesses no more significance than the volumetric change. However, the lineal shrinkage combined with the field moisture equivalent offers a means of estimating the shrinkage limit which may be used when the first two values are known and the shrinkage limit has not been determined by test.

The relation between lineal shrinkage, field moisture equivalent, and shrinkage limit computed by equations 19 and 20 for soils having a specific gravity of 2.65 is shown graphically in Figure 51. According to this figure a soil having a lineal shrinkage of 17 combined with a field moisture equivalent of 82 would have a shrinkage limit of approximately 30.

The relation between shrinkage limit and shrinkage ratio for specific gravities varying from 2.25 to 2.95 is shown in Figure 52.

TEST CONSTANTS AND MECHANICAL ANALYSES STATISTICALLY RELATED

In addition to knowing the interrelationships between the test constants it may prove helpful also to have some conception of the average relations existing between the clay, silt, and sand contents of soils and their test constants.

The relations represented by curves 3, 5, 9, and 11 (fig. 36) are, in general, those which have been reported previously as "statistical" relationships between the averages of large numbers of individual soil tests (*21*). The average mechanical analysis of soils whose tests are related according to these statistical laws is shown in Figure 53 as a function of the liquid limit.

Both the grading represented in Figure 53 and the constants represented by curves 3, 5, 9, and 11 may be considered as characteristic of "average soils," and this fact may serve as a basis for estimating the relative degree to which particular soils possess certain characteristics.

An "average" or "statistical" soil containing 72 per cent clay and no sand has, according to Figures 53 and 36, constants as follows: Liquid limit, 100; plasticity index, 54; shrinkage limit, 11; centrifuge moisture equivalent, 72; and field moisture equivalent, 45. If a soil being investigated contains 72 per cent of clay and no sand, but the constants have the values, liquid limit, 50, plasticity index, 30, shrinkage limit, 15, centrifuge moisture equivalent, 60, and field moisture equivalent, 55, its constants may be expressed as ratios of the constants possessed by the statistical soils.

As the investigations in progress disclose more and more the soil constituents which cause the constants of

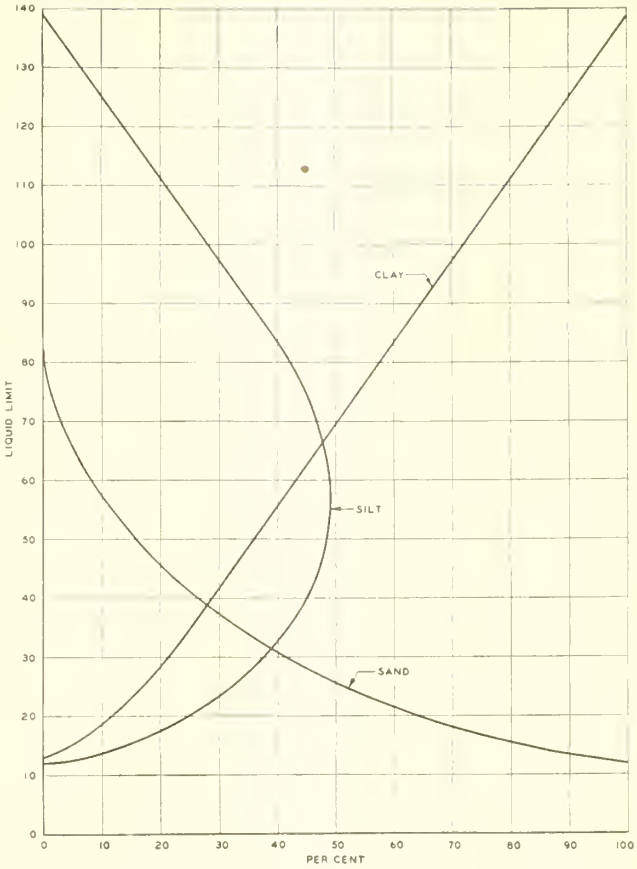


FIGURE 53.—AVERAGE MECHANICAL ANALYSIS OF SOILS WHOSE TESTS ARE RELATED ACCORDING TO CURVES 3, 5, 9, AND 11, FIGURE 36

various soils to differ from the constants of statistical soils, the relations discussed above will serve to reveal more accurately the constituents of which the soils being investigated are composed.

It is interesting to note that the representative soil constituent, clay, in Table 5, is for all practical purposes an "average" soil.

The constants of statistical soils containing both clay and sand in amounts equal to those indicated in Figure 53 are obtained in the same manner as the constants of statistical soils containing clay and no sand.

Thus the constants of a statistical soil containing 36 per cent clay and 16 per cent sand have the following values: Liquid limit, 50; plasticity index, 23; shrinkage limit, 13; centrifuge moisture equivalent, 36; and field moisture equivalent, 32.

It may prove helpful also to be able to estimate what might be termed the average influence exerted by sand admixtures upon the constants possessed by soils. The conversion curves of Figures 54 and 55 furnish a means of making estimates of this character. These curves are based on tests performed upon soils containing sand admixtures in different amounts. The sand content, referred to in these figures as "per cent sand," is expressed as a percentage of the combined weights of both sand admixture and soil.

Estimates furnished by means of conversion curves decrease in accuracy as the amount of the assumed sand admixture is increased. This is due to the fact that the curves indicate the average influence exerted by a number of sands and not the influence exerted by a particular sand. As admixtures of a particular sand

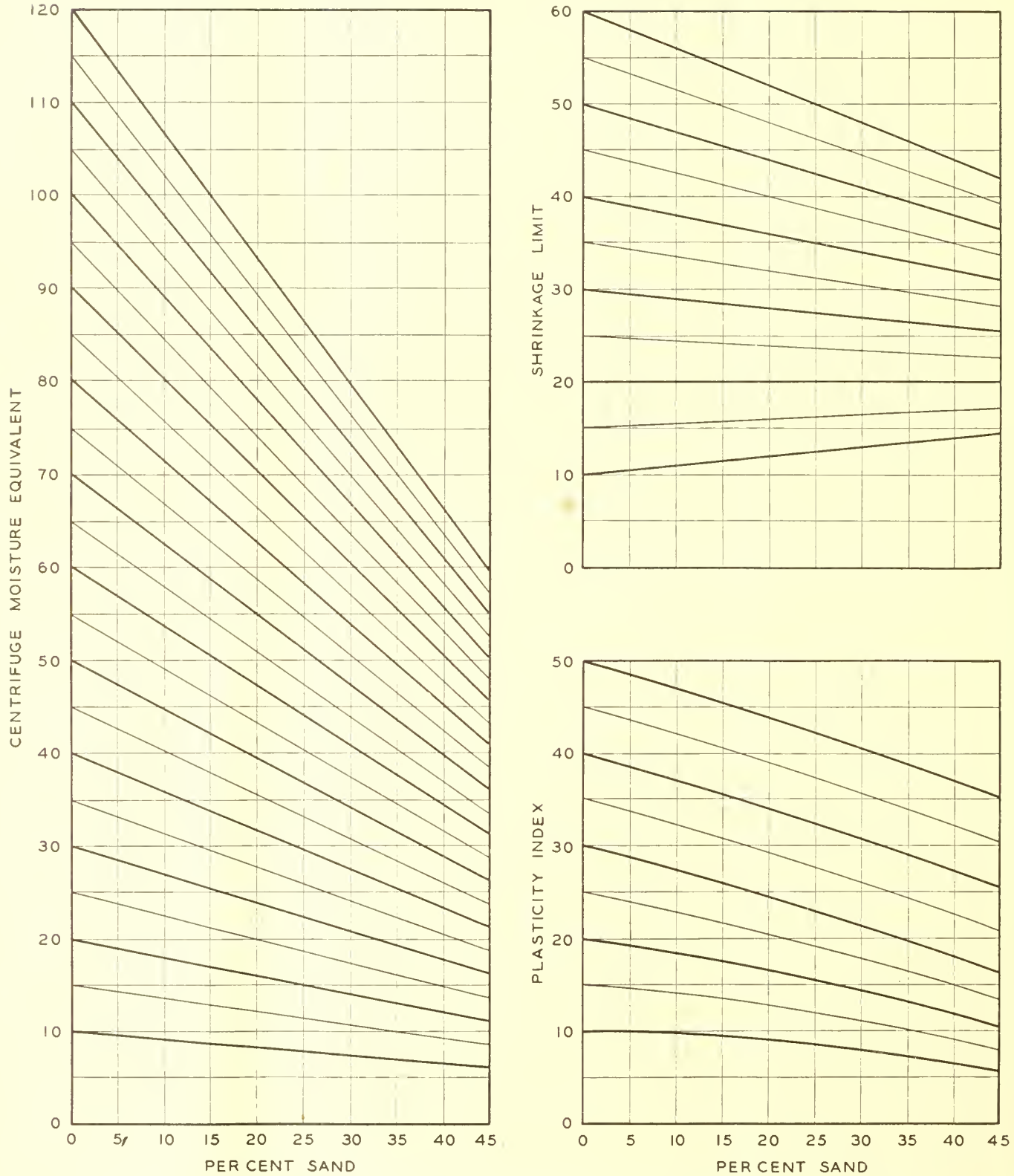


FIGURE 54.—CONVERSION CHARTS FOR DETERMINING EFFECT OF SAND ADMIXTURES ON SOIL TEST CONSTANTS

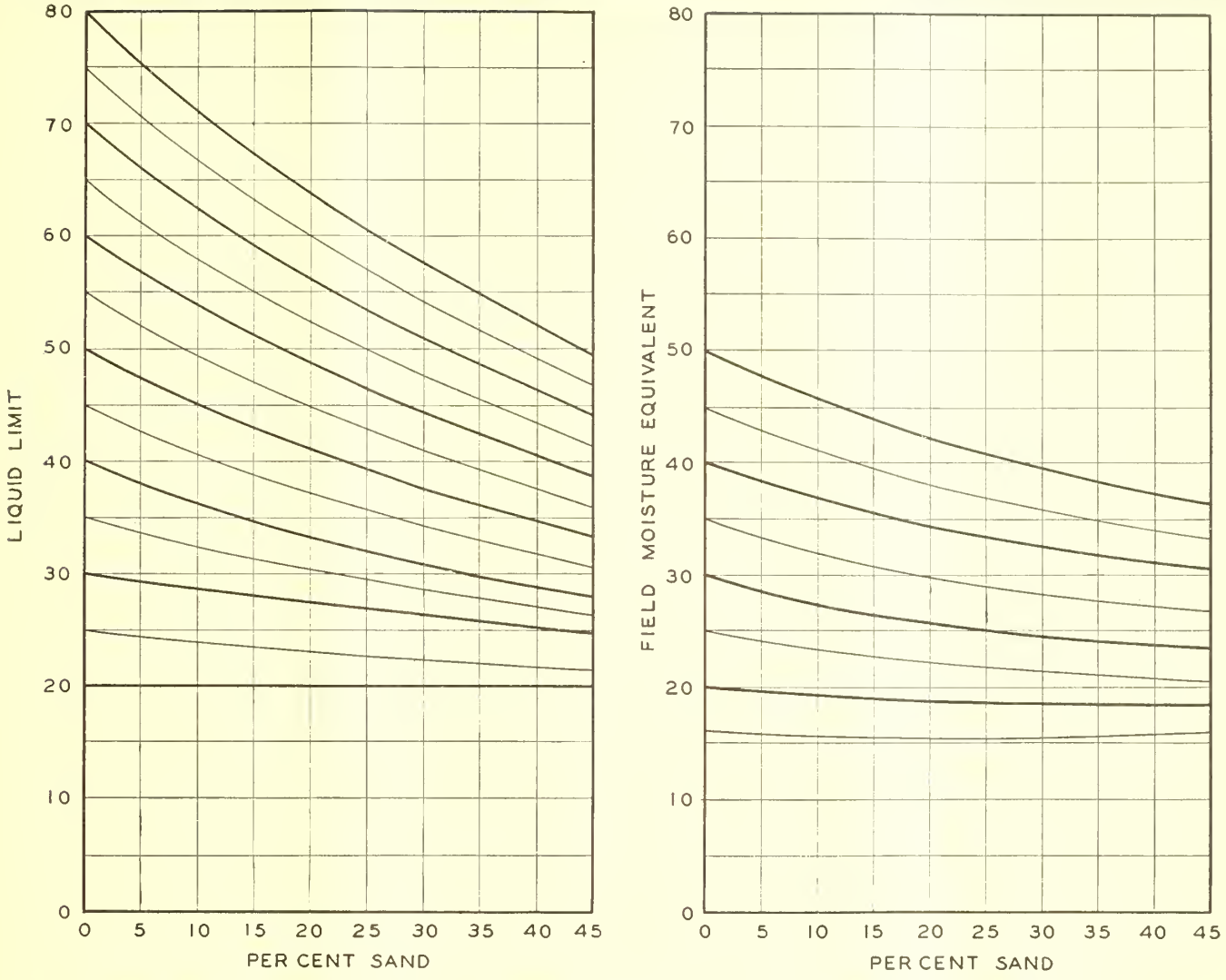


FIGURE 55.—CONVERSION CHARTS FOR DETERMINING EFFECT OF SAND ADMIXTURES ON SOIL TEST CONSTANTS

increase in amount, its individual characteristics, which depend on its grading, the size and shape of its grains, etc., will exert an increasingly important influence upon the test constants of the mixture, and cause them to vary from the estimates furnished by the conversion curves. This is especially true when the sand admixture increases in amount above about 45 per cent. As the sand admixture decreases in amount below about 45 per cent, the physical characteristics of the sand exert less and less influence on the soil test results.

In order to illustrate the use of Figures 54 and 55 let us assume that a soil sample has constants equal to those designated for sample A, Table 7. It is desired to estimate the influence exerted by adding sand in amount equal to 25 per cent of the weight of the soil. The assumed admixture in this case equals $\frac{25 \times 100}{125}$ per cent or 20 per cent of the resulting combination of sand and soil. Consequently, we reduce each constant of sample A by an amount indicated in the corresponding conversion diagram for a sand admixture of 20 per cent. The resulting constants are given opposite sample X in Table 7.

TABLE 7.—*Test constants estimated on the basis of the conversion curves for sand admixtures*

Sample	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
A	45	19	20	1.7	33	30
X	37	16	20	[1] 1.7	26	26

[1] Specific gravity assumed to equal that of sample A. Therefore, with equal shrinkage limits the shrinkage ratios will be equal.

PART III: UTILIZATION OF THE SUBGRADE SOIL IDENTIFICATION CHART

The soil identification chart, shown in revised form in Figure 56 serves a triple purpose. First, it offers a means of identifying those subgrade soils whose performance in service has been learned; second, it assists in predicting the performance of soils comprising the subgrades of roads to be constructed; and, third, it assists in disclosing the influence exerted by either physical or chemical admixtures upon the performance of subgrade soils.

In order to demonstrate how these purposes are accomplished the improvement of soils by means of admixtures is illustrated and a limited number of soils belonging to the different groups are analyzed with respect to their constants. To facilitate this demonstration the basic requirements of good sand clay roads are discussed and both the grading and the constants indicative of soils of the different groups are reviewed.

GRADING OF GOOD TOP SOILS DEPENDS UPON THE CHARACTER OF THE BINDER

Theoretically stable mixtures consist of a well graded coarse material (grains larger than about 0.05 millimeter in diameter) possessing high internal friction, and a binder. The binder, which may be visualized as occupying the sand pores, should have sufficient cohesion to cement the sand grains together; and, upon wetting, the binder should expand in amount just sufficient to close the surface pores and thus prevent water from penetrating and softening the interior of the road surface. When the binder expands an amount greater than that required to close the sand pores, the sand grains are likely to become unseated, thus reducing the stability of the mixture. When the binder does not expand sufficiently to close the sand pores, water may enter and soften the road surface. It follows that the amount of binder required to furnish stable mixtures depends upon the expansion properties of the binder. Binders which are only slightly expansive may be used in an amount sufficient to fill the pores of the sand almost completely. As the expansive properties of the binder become more and more important, the amount used without danger of unseating the sand grains must of necessity become smaller and smaller.

Of two soils whose tendency to shrink or expand are equal, the one having the greater amount of cohesion should be the better binder. Of two soils having equal cohesion, that having the less tendency to shrink or expand should be the better binder, since a greater amount of it can be used than of the more expansive soil.

This theoretical conception that the amount of binder required depends upon the characteristics of the binder is substantiated by Doctor Strahan's (*22*) studies of roads in service. He emphasizes the fact that while soil mixtures having particular gradings are likely to produce stable wearing courses, the gradings are a qualitative rather than a quantitative measure of efficiency. The cohesive and shrinkage properties of the fine material are of utmost importance. Doctor Strahan reports kaolin as being an exceptionally good binder; and according to its constants kaolin possesses properties theoretically required by good binders. These constants are:

Liquid limit	60
Plasticity index	26
Shrinkage limit	36
Shrinkage ratio	1.3
Centrifuge moisture equivalent	49
Field moisture equivalent	36

The liquid limit of 60, for instance, combined with a plasticity index of 26 indicates cohesive properties approaching those of an inert clay (curve 3). The shrinkage limit of 36 equals that of elastic silts and muck (curve 6). The centrifuge moisture equivalent of 49 indicates a water capacity slightly greater than that of average soils (curve 9). The field moisture equivalent of 36 indicates a resistance to water penetration characteristic of colloidal soils (curve 11): and the shrinkage limit being equal to the field moisture equivalent, the lineal shrinkage, like that of sand, is equal to 0.

It is clear that kaolin has both cohesion and water-retentive properties in moderate amount, relatively high resistance to water penetration when at the shrinkage limit, and negligible shrinkage properties. The constants possessed by kaolin may, therefore, serve as a basis for identifying good binders. These characteristics, it will be noted, indicate a plastic variety of the Group A–5 subgrade with an exceptionally low field moisture equivalent.

SUBGRADE SOILS MAY BE IMPROVED BY ADMIXTURES

It is natural that efforts should be made to increase the stability of certain varieties of subgrade soil by admixtures of suitable materials. The success of such efforts depends upon the manner and the extent to which unstable mixed materials differ in character from those which are stable. The changes which admixtures produce in the test constants indicate their effect on the physical properties of the soil.

If, according to mechanical analysis, the soil is deficient in coarse material and has appreciable plasticity, admixtures of coarse, granular materials, such as sand, slag, gravel, or crushed stone, may prove beneficial. It has been observed that the efficiency of rounded gravel may be increased considerably by either crushing or adding angular fragments to the rounded material.

If the active portion of the soil mortar, because of domination of clay, is high in both plasticity and shrinkage properties, admixtures of either porous silt or materials such as hydrated lime, diatoms, etc., having high shrinkage limits may serve to reduce the shrinkage properties without reducing too much the plasticity of the soil. Penetrative bituminous materials applied to reduce the moisture capacity of the clay may prove beneficial.

TABLE 8.—*Test results on several soils combined with various admixtures*

Soil No.	Admixture	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent		Volumetric change
				Limit	Ratio	Centrifuge	Field	
		P. ct.	P. ct.	P. ct.	P. ct.	P. ct.	P. ct.	P. ct.
3,770	None	24	0	20	1.7	15	24	
	60 per cent A sand [1]	25	0			10	25	
	20 per cent mica	44	0	38	1.2	23	44	
	15 per cent peat	37	0	28	1.3	26	34	
	15 per cent diatoms	73	38	36	1.3	38	42	
	6 per cent colloids	39	20	16	1.6	32	28	
4,056	None	66	40	11	2.1	[2] 53	29	
	60 per cent A sand	36	20	17	1.9	23	27	
	20 per cent mica	70	42	16	1.8	63	51	
	15 per cent peat	72	38	15	1.8	66	47	
	15 per cent diatoms	89	54	28	1.6	75	52	
	6 per cent colloids	89	62	10	2.1	[2] 133	40	
5,041	None	65	36	14	1.9	55	50	68.4
	15 per cent hydrated lime	84	58	24	1.6	59	42	28.8

[1] Angular sand consisting of crushed diabase.
[2] Waterlogged.

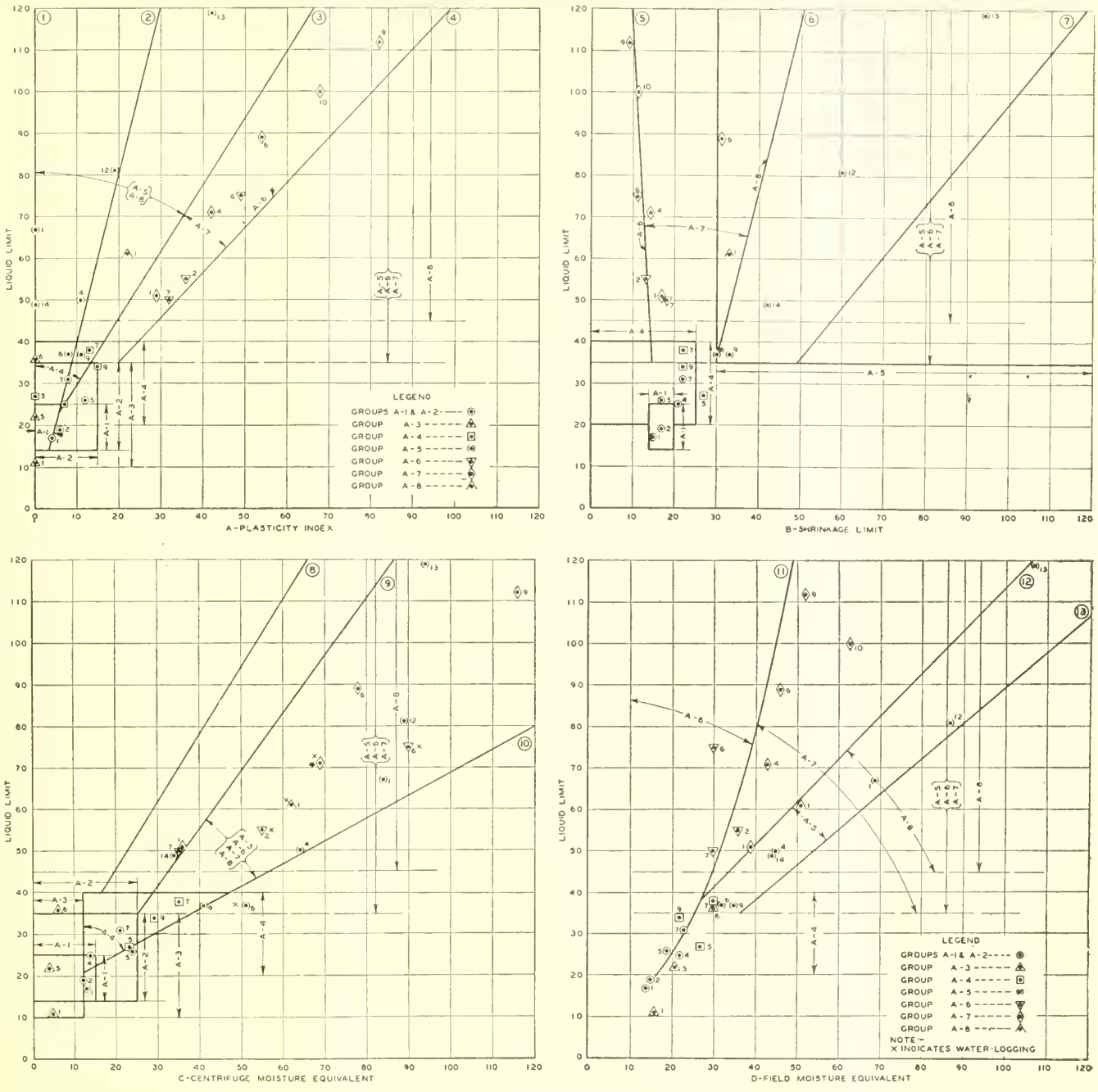


FIGURE 56.—THE SOIL IDENTIFICATION CHART

If the soil contains an abundance of coarse material and lacks clay and silt, cohesive materials obviously must be added. In this case either a proper clay binder may be added or the cohesionless coarse material may be treated with a highly penetrative bituminous material and covered with a light application of granular material.

If the soil, according to mechanical analysis, has proper grading but is low in both plasticity and shrinkage, and is to be placed on a very dry subgrade, the addition of a cohesive material may prove beneficial. Thus the gluey colloids which are detrimental to soil in large amounts may prove beneficial when present in very small amounts. Bentonite, for instance, added in the laboratory to a fine sandy loam in amounts not exceeding 3 per cent has the effect of introducing plasticity and resistance to erosion without increasing the shrinkage in detrimental amounts. Admixtures of bituminous materials, referred to above, may also serve this purpose very efficiently.

The data contained in Table 8 illustrate how the test constants disclose the influence exerted by different kinds of admixtures upon the characteristics of soils.

It is interesting to note that when mica and diatoms are added to the nonplastic soil, No. 3,770, both the shrinkage limits and the field moisture equivalents are very appreciably increased. The same is true when diatoms are added to the plastic soil, No. 4,056. When mica is added to the plastic soil only the field moisture equivalent is very appreciably increased.

It is interesting also to note that the volumetric change of soil No. 5,041 is 68.4 and that of the mixture of this soil and hydrated lime is only 28.8.

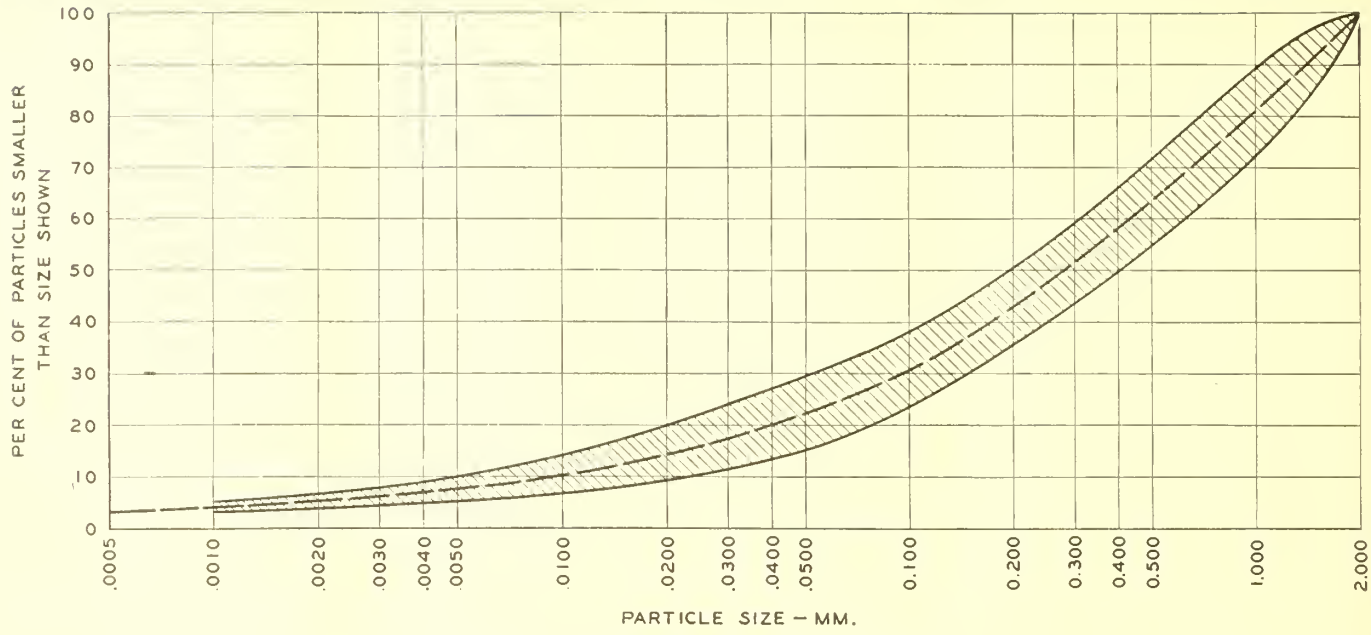


FIGURE 57.—GRADING OF GOOD SOIL MORTARS

GRADING AND CONSTANTS OF SUBGRADE GROUPS REVIEWED

The gradings of the various subgrade groups and the values of the test constants characteristic of them are repeated here as an aid to the discussion of soil identification, with which this part of the report is chiefly concerned.

Group A–1.—Grading: Material retained on the No. 10 sieve not more than about 50 per cent. The soil mortar, that fraction passing the No. 10 sieve, to consist of clay, 5 to 10 per cent; silt, 10 to 20 per cent; total sand, 70 to 85 per cent; and coarse sand, 45 to 60 per cent. Average effective size approximately 0.01 millimeters and uniformity coefficient greater than 15. The band shown in Figure 57 illustrates graphically the grading of good soil mortars.

Constants: Liquid limit not less than 14 nor greater than 25; plasticity index approximately equal to that indicated by curve 2 (fig. 56, A) and seldom greater than 8; shrinkage limit seldom less than 14 or greater than 20; and centrifuge moisture equivalent not apt to be greater than 15.

Fraction passing the No. 200 sieve—see constants of kaolin, p. 134, and Group A–5 subgrade below.

Group A–2.—Grading: Not less than about 55 per cent of sand in the soil mortar.

Constants: Liquid limit generally not less than 14 or greater than 35; a plasticity index of zero with a significant shrinkage limit or a plasticity index greater than zero and less than 15 with or without a significant shrinkage limit; centrifuge moisture equivalent not greater than 25.

Group A–3.—Grading: Effective size not likely to be less than 0.10 millimeters.

Constants: Liquid limit not appreciably greater than 35; no plasticity index; no significant shrinkage limit; centrifuge moisture equivalent less than 12.

Ability of sands to resist sliding when wet indicated as follows: Liquid limits of 10 to 14 signify beach and other rounded sands which slide easily; liquid limits of 30 to 35 indicate rough angular particles which do not slide easily. In addition, liquid limits when lower than field moisture equivalents indicate materials which flow under partial saturation; when equal to the field moisture equivalents, the liquid limits indicate average sands which flow under full hydrostatic uplift. Liquid limits greater than field moisture equivalents indicate rough-grained sands which flow only when in a state less consolidated than that represented by the field moisture equivalent. (See fig. 58.)

Group A–4.—Grading: Less than 55 per cent sand.

Constants: Liquid limit seldom less than 20 or greater than 40; plasticity index not greater than those indicated by curve 3; shrinkage limit not likely to be greater than 25; centrifuge moisture equivalent approaching those indicated by curve 10, between 12 and 50; when greater than liquid limit indicates varieties of soils inclined to be especially unstable in the presence of water; field moisture equivalent equal to or somewhat greater than those indicated by curve 11, with a maximum of about 30.

Increase in expansive properties generally indicated when shrinkage limits exceed 20 and approach those represented by curve 6; especially likely when field moisture equivalent exceeds centrifuge moisture equivalent.

Group A–5.—Grading: Less than 55 per cent sand. (Exceptions occur.)

Constants: Liquid limit usually greater than 35; plasticity index seldom greater than those indicated by curve 3; centrifuge moisture equivalent greater than 12, often lying between curves 9 and 10; not likely to water-log. (Exceptions occur.)

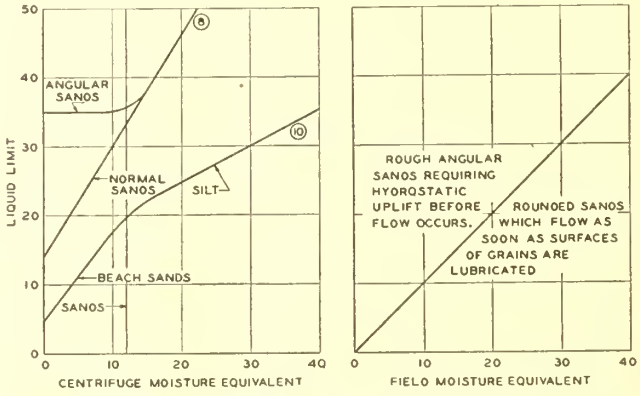


FIGURE 58.—SIGNIFICANT RELATIONS BETWEEN TEST CONSTANTS FOR GROUP A–3 SUBGRADE SOILS

Shrinkage limit generally greater than 30 and greater than 50 for very undesirable members of this group. May approach values indicated by curve 6 for silts containing peat and approach those indicated by curve 7 for soils containing either diatoms or mica in appreciable amount. Field moisture equivalent approaching those indicated by curve 12 for silts containing peat in appreciable amount and those indicated by curve 13 for highly elastic soils containing mica or diatoms in appreciable amount. The kaolins, representing good binders, are members of group possessing relatively high plasticity indices and low field moisture equivalents.

Group A–6.—Grading: Seldom contains less than 30 per cent clay.

Constants: Liquid limit usually greater than 35; plasticity index approximately represented by curve 4; shrinkage limit not likely to be appreciably greatr than that indicated by curve 5; centrifuge moisture equivalent test generally productive of water-logging; likely to lie between curves 9 and 10; field moisture equiva-

lent seldom exceeding those indicated by curve 11, but may be appreciably less for certain colloidal soils. Volumetric change generally greater than 17.

Group A–7.—Grading: Seldom contains less than 30 per cent clay.

Constants: Liquid limit usually greater than 35; plasticity index varies between those indicated by curves 3 and 4; shrinkage limit generally varies between those indicated by curves 5 and 6; centrifuge moisture equivalent varies between those indicated by curves 9 and 10; water-logging in centrifuge test may not occur even at very high moisture equivalents. Field moisture equivalent greater than those indicated by curve 11. Relatively low shrinkage limits with high field moisture equivalents indicate presence of colloidal organic matter. Relatively high shrinkage limits indicate the possibility of frost heave.

Group A–8.—Grading: Not significant.

Constants: Liquid limit greater than 45; plasticity index generally less than those indicated by curve 3; shrinkage limit indicated approximately by curve 6; centrifuge moisture equivalent between curves 9 and 10; field moisture equivalent likely to be greater than those indicated by curve 12.

Water-logging in the centrifuge test is characteristic of the mucks containing clay and colloids, whereas very high equivalents without water-logging are characteristic of peat not more than slightly decomposed.

NOMENCLATURE AND FORMULAS LISTED

The equations of the relationship curves 1 to 13, inclusive, and the basic formulas which have been developed in this report are listed below. The nomenclature for these equations is as follows:

$P.\ I.$ = plasticity index.
$L.\ L.$ = liquid limit.
$S.$ = shrinkage limit.
$C.\ M.\ E.$ = centrifuge moisture equivalent.
$F.\ M.\ E.$ = field moisture equivalent.
e = voids ratio.
e_o = voids ratio of dry sample.
V_v = volume of voids.
V_s = volume of soil particles.
V = volume of wet soil sample.
V_o = volume of dry soil sample.
C_o = volume change, percentage of V_o.
C_f = volumetric change, from $F.\ M.\ E.$, percentage of V_o.
$L.\ S.$ = lineal shrinkage, percentage of wet length.
M_w = weight of moisture.
W = weight of wet sample.
W_o = weight of dry sample.
w = moisture content, percentage of W_o.
w_v = moisture content, percentage of V_s.
G = specific gravity of soil particles.
P = porosity.
h = height of capillary rise in centimeters.
r = radius of capillary tube in centimeters.
a = width of pores in centimeters.
$S.\ F.$ = shrinkage force in grams per square centimeter.

EQUATIONS OF CURVES FOR SOIL IDENTIFICATION CHART

Curve 1.—$P.\ I. = 0$

Curve 2.—$P.\ I. = 0.25\ L.\ L.$

Curve 3.—$P.\ I. = \dfrac{L.\ L. - 14}{1.60}$

Curve 4.—$P.\ I. = \dfrac{L.\ L. - 14}{1.07}$

Curve 5.—$S = 21 - 1.1\sqrt{L.\ L. - \dfrac{L.\ L.^2}{800}}$

Curve 6.—$S = \dfrac{L.\ L. + 86}{4.1}$

Curve 7.—$S = \dfrac{L.\ L. + 26}{1.24}$

Curve 8.—$C.\ M.\ E. = \dfrac{L.\ L. - 14}{1.61}$

Curve 9.—$C.\ M.\ E. = 0.72\ L.\ L.$

Curve 10.—$C.\ M.\ E. = \dfrac{L.\ L. - 14}{0.55}$

Curve 11.—$F.\ M.\ E. = \sqrt{15.2\,(L.\ L. - 16.3)} + 9$

Curve 12.—$F.\ M.\ E. = \dfrac{L.\ L. - 10}{1.03}$

Curve 13.—$F.\ M.\ E. = \dfrac{L.\ L. - 4}{0.85}$

BASIC SOIL FORMULAS

$$e = \frac{V_v}{V_s} \qquad (1)$$

$$= \frac{wG}{100} \qquad (10)$$

$$P = \frac{e}{1+e} \times 100 \qquad (11)$$

$$C_o = \frac{V - V_o}{V_o} \times 100 \qquad (2)$$

$$= \frac{e - e_o}{1+e_o} \times 100$$

$$C_f = (F.\ M.\ E. - S)\ R \qquad (18)$$

$$= \frac{F.\ M.\ E. - S}{\frac{1}{G} + \frac{S}{100}} \qquad (19)$$

$$L.\ S. = \frac{1 - \sqrt[3]{\frac{100}{C_f + 100}}}{0.01} \qquad (20)$$

$$M_w = W - W_o \qquad (5)$$

$$w = \frac{M_w}{W_o} \times 100 \qquad (4)$$

$$w = \frac{W - W_o}{W_o} \times 100 \qquad (6)$$

$$V_s = \frac{W_o}{G} \qquad (7)$$

$$w_v = wG \qquad (8)$$

$$h = \frac{0.153}{r}$$ (See reference (1).)

$$S.\ F. = \frac{0.306}{a} \qquad (14)$$

$$S = w - \frac{V - V_o}{W_o} \times 100 \qquad (15)$$

$$= \frac{100\ G - 100\ R}{RG}$$

$$R = \frac{W_o}{V_o} \qquad (16)$$

$$= \frac{100\ G}{100 + SG}$$

SOIL SAMPLES ANALYZED BY GROUPS

In the pages which follow the test results from a large number of soil samples are tabulated and analyzed. With the aid of the soil identification chart, the place of each soil in the uniform subgrade classification is determined. From this procedure the reader may gain an idea of how identifications of this sort are accomplished in practice.

The plastic limit, shrinkage limit, centrifuge moisture equivalent, and field, moisture equivalent of a number of samples from each subgrade group are plotted as functions of the liquid limit on the soil identification chart, Figure 56, in order that the comparative relations of the constants may be studied in reference to the numbered curves 1 to 13.

Group A–1 and Group A–2 subgrades.—The constants and grading of samples of soil containing both coarse andfine materials are included in Tables 9 and 10, respectively.

Sample 1, which represents a stable soil from South Carolina, satisfies both the grading and the constant requirements of the Group A–1 subgrade.

TABLE 9.—*Group A–1 and Group A–2 subgrades, coarse fractions and binders. Constants of material passing the No. 40 sieve except as noted*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	17	4	15	1.9	13	14
2	19	6	17	1.8	12	15
3	20	7	18	1.8	13	16
4	25	7	21	1.7	14	22
5	26	12	17	1.8	24	19
6	25	5			23	23
7	31	8	22	1.6	21	23
8	25				19	27
9	28	13	22	1.7	16	21
10	30	15	21	1.7	20	22
9–B [1]	49	29			27	
10–B [1]	58	34			36	
9–C [2]	24				5	25
10–C [2]	23				2	27

[1] Fraction passing No. 200 sieve.
[2] Fraction passing No. 40 sieve and retained on No. 200 sieve.

TABLE 10.—*Mechanical analyses of Group A–1 and Group A–2 subgrades*

Sample No.	Gravel particles larger than 2 millimeters	Mortar				
		Coarse sand 2.0 to 0.25 millimeter	Fine sand 0.25 to 0.05 millimeter	Silt 0.05 to 0.005 millimeter	Clay below 0.005 millimeter	Colloids below 0.001 millimeter
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
Good soil mortar		[1] 45–60		10–20	5–10	3–5
1	10	51	25	16	8	6
2	1	37	33	18	12	5
3	1	21	58	6	15	12
4	3	26	45	16	13	9
5	66	31	29	18	22	15
6	0	1	87	5	7	5
7	8	36	30	26	8	3
8	0	45	29	16	10	4
9	50	37	39	3	21	18
10	48	53	19	5	23	20

[1] Total sand 70 to 85 per cent.

Sample 2, which was obtained from a soil performing satisfactorily as fill in Escambia Bay, Fla., satisfies the constants but not the grading requirement of a Group A–1 material.

Its low centrifuge moisture equivalent of 12 indicates that the slight excess of clay above that generally indicating A–1 mortars is not likely to prove detrimental; and since the coarse sand in which this soil is deficient functions primarily to furnish the hardness required in wearing surfaces, this soil would probably prove highly stable as a subgrade.

The same is true for both sample 3, which was taken from a soil serving as a road surface in South Carolina, and sample 4, which was taken from a soil moderately stable when used as an untreated road surface in Madison County, Va., and highly stable when covered with bituminous surface treatments. (See fig. 59.)

FIGURE 59.—SURFACE-TREATED ROAD OF GROUP A–2 MATERIAL IN MADISON COUNTY, VA.

Sample 5, obtained from gravel used as road surface in Oklahoma; sample 6, from soil found by W. H. Dumont, of the United States Bureau of Fisheries, to be very unstable when occurring as bottom in Deep Creek, Ga.; and sample 7, from soil productive of frost heave in New Hampshire, fail to satisfy either the grading or the subgrade constant requirements of the Group A–1 material.

Sample 8, representing the Florida limerocks, satisfies the grading but not the constant requirements of the Group A–1 subgrade.

The high clay content of sample 5 is disclosed by the correspondingly high plasticity index of 12 and the centrifuge moisture equivalent of 24. The high plasticity index might indicate a desirable property, but the relatively high centrifuge moisture equivalent suggests water retentive properties not productive of stability. This detrimental property is probably offset to some extent by the fact that the field moisture equivalent is appreciably less than the centrifuge moisture equivalent.

The high silt content of sample 7 has produced a relatively high centrifuge moisture equivalent without raising the plasticity index in corresponding amount, one of the characteristics of soils productive of frost heave.

Special attention is called to sample 6, which, when submerged in water, has the properties of quicksand. This fact is disclosed by the nonuniform grading, which shows that the sample consists of 87 per cent fine sand, and the relatively high water-retentive property. The centrifuge moisture equivalent of this sample, containing but 12 per cent of particles smaller than 0.05 millimeter in diameter is practically the same as that of sample 5, which contains 40 per cent of particles smaller than 0.05 millimeter in diameter, and, furthermore, equals the field moisture equivalent which, in this case, can be assumed to indicate the amount of water required to fill the pores of the soil completely.

Sample 8 represents the nonplastic variety of the shell rocks. Tests performed on small beams of this material disclosed that when thoroughly dry this variety of limerock has practically no cohesion. Therefore, the stability of this material when used as a road surface must be due to the cohesion furnished by capillary pressure. Figure 60 shows how limerock occurs in nature.

ALTERNATE METHOD OF INVESTIGATING GRADED MATERIALS DISCUSSED

A somewhat elaborate method suggested for investigating the properties of mixed materials consists of testing the mixture as such and also testing separately both the nonexpansive and expansive soils of which it is composed, and thus investigating soil mixtures in a manner similar to that employed when investigating bituminous mixes, concrete, or other materials consisting of a binder and an aggregate.

A procedure of this kind includes (*a*) the determination of the relative resistance furnished by the soil mortar to water absorption by means of slaking tests; (*b*) the determination of the relative strength of the mortar when dried, by means of a crushing or impact test; (*c*) the determination of the grading of the soil mortar by means of the combined sieve and hydrometer method; (*d*) the determination of both the plastic and shrinkage properties of that fraction of the material passing the No. 200 (0.074 millimeter) sieve by means of the test constants; and (*e*) the determination of the character of that fraction of the sample retained on the No. 200 sieve according to the procedure employed in identifying the characteristics of Group A–3 subgrades discussed subsequently.

Samples 9 and 10, Tables 9 and 10, representing, respectively, a stable and an unstable soil in South Carolina, serve to illustrate how coarse and fine materials may be investigated separately.

Table 10, for instance, discloses that sample 9, compared with the average good soil, contains an excess of clay and fine sand and is deficient in silt and coarse sand. Sample 10, compared with sample 9, contains a greater amount of coarse and a less amount of fine sand.

The binders of the two samples, 9–B and 10–B, Table 9, are similar in character, although the constants of 10–B are higher than those of 9–B. Both, however, have plasticity indices greater than that of kaolin (greater than that indicated by curve 3), combined with relatively low water retentive properties (curve 9). This indicates open-structure clays unlikely to shrink or expand in appreciable amount.

The constants of the sand fractions 9–C and 10–C, Table 9, are not radically different. Sample 9–C, however, which contains the greater amount of fine sand, has the higher centrifuge moisture equivalent. The liquid limit of sample 9–C is practically equal to the amount of water (field moisture equivalent) required to saturate the sample completely, whereas the liquid limit of sample 10–C is reached at a moisture content slightly below that required to saturate the soil completely.

From these evidences it would seem that the excellent quality of the binder primarily accounts for the fact that the soil represented by sample 9 can contain so large a clay content and yet remain stable.

That the same high clay content proved detrimental to the soil represented by sample 10 may be explained by the fact that, because of difference in grading, the surface area of the grains in the sand sample 9 may exceed by more than 40 per cent that of the grains in the sand fraction sample 10. Therefore, if the colloidal material, approximately the same in both samples, can be considered as a glue which coats the surfaces of the sand grains, sample 9, with the larger surface area, will cause the glue to be distributed in films thinner than those in sample 10. For this reason the quantity of glue (colloidal material) which causes the films to be excessively thick in sample 10 may not prove detrimental to sample 9.

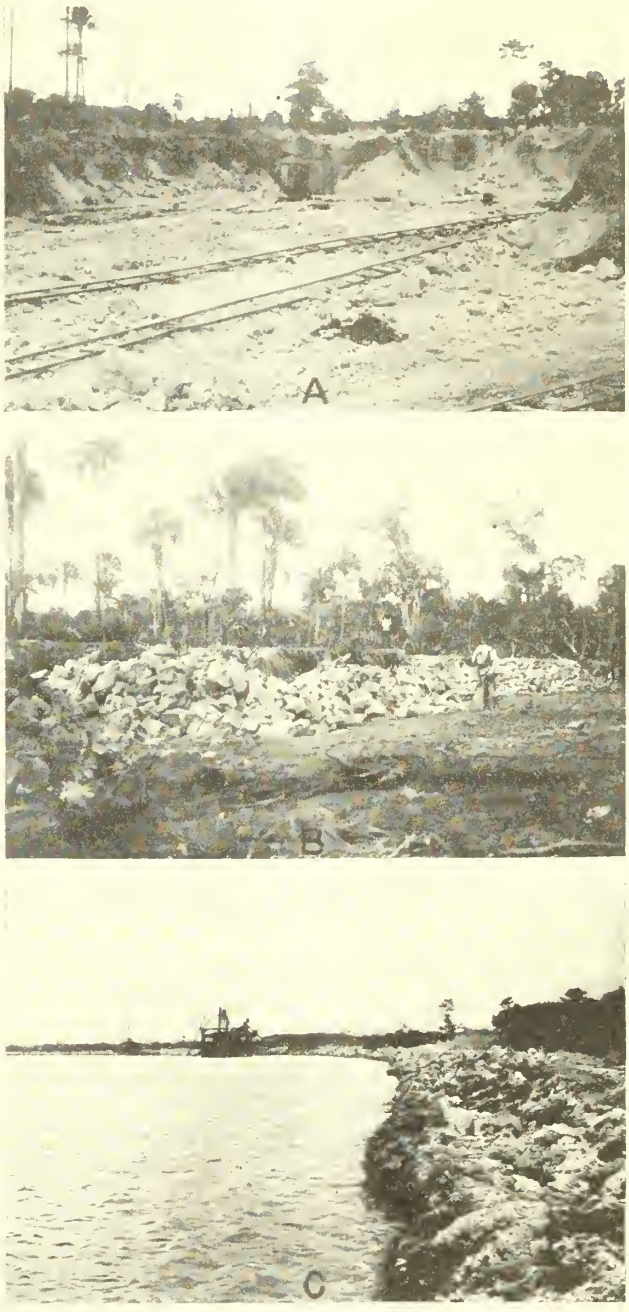


FIGURE 60.—EXAMPLES OF SHELL ROCK IN NATURAL LOCATION: A, CHALKY LIMEROCK OF THE OCALA, FLA., REGION. B, COCHINA ROCK NEAR ORMOND BEACH, FLA. C, OGUS ROCK, LOWER EAST COAST OF FLORIDA

Group A–3 subgrades.—The samples included in Tables 11 and 12 satisfy in both grading and constants the requirements of the cohesionless sand subgrades which provide good drainage and which do not have the tendency to heave under frost action.

TABLE 11.—*Group A–3 subgrades; Constants of material passing the No. 40 sieve*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	11	0			5	16
2	20	0			4	25
3	19	0			3	17
4	20	0			3	21
5	22	0			4	21
6	36	0			6	30

TABLE 12.—*Mechanical analyses of Group A–3 subgrades*

Sample No.	Gravel particles larger than 2.0 milimeters	Coarse sand 2.0 to 0.25 milimeter	Fine sand 0.25 to 0.05 milimeter	Silt 0.05 to 0.005 milimeter	Clay below 0.005 milimiter	Colloids below 0.001 milimeter
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	0	74	18	3	5	3
3	0	25	72	1	2	1
4	0	20	78	1	1	0
5	0	41	56	2	1	0

FIGURE 61.—THREE-INCH GROUTED BRICK ROAD CONSTRUCTED ABOUT 1916 ON GROUP A–3 SUBGRADE IN FLORIDA. STILL IN SERVICE. CONCRETE SHOULDERS CONSTRUCTED IN 1921

Sample 1 was obtained from a Florida sand which serves excellently as subgrade, when prevented from flowing laterally, for relatively thin road surfaces. (See figs. 61 and 62.) Sample 2 represents that fraction of Potomac River sand passing the No. 20 and retained on the No. 100 sieve. Sample 3 was obtained from a California sand which serves excellently as a subgrade for concrete pavements 4½ inches thick (*23*). Sample 4 was taken from a Minnesota sand which becomes highly stable when treated with bituminous materials possessing penetrative properties in high degree, and covered with a thin application of granular material. Sample 5 is a New Hampshire sand which furnishes excellent support when treated in a manner similar to that described for sample 4. Sample 6 represents that fraction of crushed diabase passing the No. 20 and retained on the No. 100 sieve.

According to the significance of the relation existing between the liquid limits and the centrifuge moisture equivalents of these soils, the soils represented by sample 1 would be expected to have very low stability and those represented by sample 6 would be expected to have very high stability in the presence of water.

Additional information on the character thees of soils is furnished by the relation between the liquid limit and the field moisture equivalent. Assuming that the field moisture equivalents of sand equal the amounts of water required to fill the pores of the sands completely when compressed by a very small but constant pressure, the glacial sands represented by samples 4 and 5, Table 11, are likely to flow when completely saturated (full hydrostatic uplift). The beach and river sands of samples 1 and 2 will flow when the grains are lubricated, rather than as a result of full hydrostatic uplift. The angular fragments of sample 6 require water in amounts greater than the field moisture equivalent to cause flow.

FIGURE 62.—SURFACE TREATED LIMEROCK ROAD SIX INCHES THICK IN FLORIDA

The fact that the liquid limit is greater than the field moisture equivalent indicates that, in order to flow, the angular particles must exist in a state looser than that represented by the field moisture equivalent. It appears from these facts that sample 1, when in the presence of water, is likely to be the least, and sample 6 the most stable of the sands listed in Tables 11 and 12.

Group A–4, subgrades.—Samples 1 to 9, inclusive, Tables 13 and 14, have constants indicative of those soils which have the tendency to heave under frost action and to lose stability as a result of water absorption even when not manipulated.

Samples 1 and 2 were made up, respectively, from commercial rotten stone and chalk. Although these materials are not natural soils they have the properties common to the A–4 subgrades. Sample 3 was composed of marl from Florida. Marl when kept dry and not subjected to frost action serves excellently as base course material. Samples 4 and 5 were obtained from New Hampshire silts which have been observed to heave in detrimental amounts under frost action. Sample 6 was obtained from a silt found in Missouri. Pavements laid on this soil have been observed to crack in appreciable amount. Samples 7 and 8 represent soils in the Minnesota frost-boil area which heave under frost and lose stability during the spring thaws. (See fig. 63.) Sample 9 was composed of lithopone, which has been found by Prof. Stephen Taber to suffer important frost heave.

In grading this soil should belong to the highly colloidal clays of either Group A–6 or A–7. The plasticity index of 15 compared with the liquid limit of 34 is slightly higher than those designating the A–4 subgrades. The relatively high shrinkage limit and low field moisture equivalent, indicating that this material is unlikely to shrink in appreciable amount, suggests the performance of silt instead of clay. Com-

TABLE 13.—*Group A–4 subgrades; constants of material passing the No. 40 sieve*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	32	11	22	1.7	[1] 39	24
2	29	6	28	1.5	34	26
3	38	12	24	1.6	37	30
4	25	0	25	1.6	19	24
5	27	0	27	1.6	23	27
6	32	10	24	1.7	29	28
7	38	13	22	1.6	35	30
8	32	14	22	1.7	33	27
9	34	15	22	2.0	29	22

[1] Water-logged.

TABLE 14.—*Mechanical analyses of Group A–4 subgrades*

Sample No.	Gravel particles larger than 2.0 millimeters	Coarse sand 2.0 to 0.25 millimeters	Fine sand 0.25 to 0.05 millimeter	Silt 0.05 to 0.005 millimeter	Clay below 0.005 millimeter	Colloids below 0.001 millimeter
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	0	0	7	57	36	17
2	0	0	13	37	50	3
4	1	14	34	28	24	1
5	0	13	21	42	24	2
6	0	5	10	58	27	12
7	1	7	20	52	21	4
8	1	9	26	46	19	4
9	0	0	2	3	95	67

parison of the constants of this material with those of statistical soils also discloses its inactivity.

A statistical soil, for instance, containing 95 per cent clay has a liquid limit of 132, a plasticity index of 74, a shrinkage limit of 10, a centrifuge moisture equivalent of 95, and a field moisture equivalent of 51. Thus, lithopone, irrespective of its grading, has relatively very low cohesive properties, and this fact, combined with its negligible shrinkage properties and a liquid limit smaller than 40, causes lithopone to be grouped with the silts.

Group A–5 subgrades.—The constants and the grading characteristic of Group A–5 material are shown in Tables 15 and 16. In grading, these samples contain either clay or silt in dominating amounts. Only one sample has a liquid limit smaller than 35; all samples have plasticity indices smaller than those indicated by curve 3 and all but two samples have shrinkage limits equal to or greater than 30. The four samples whose shrinkage limits are not given have values of this quantity higher than the liquid limit.

Samples 1, 2, and 3 were made, respectively, from commercial hydrated lime, pumice, and talc. Samples 4 and 5 were made from barium sulphate and reground quartz, respectively. Professor Taber found the former to heave very appreciably and the latter to heave but very little as a result of frost, in experiments performed in the laboratory. Samples 6 and 7 were composed of silt found in St. Peter, Minn., which, according to F. C. Lang, heaved several feet during the winter of 1928–29. Samples 8 and 9 were obtained from New Hampshire silts which have been found by W. F. Purrington to heave in appreciable amount under frost action. Samples 10 and 11 were made up from Oregon silts which have been observed by R. H. Baldock to suffer important frost heave.

In grading, samples 6, 7, and 8, which are comparable to barium sulphate (sample 4), contain clay in dominating amount. Samples 9, 10, and 11, which are comparable to reground quartz (sample 5), contain silt in dominating amounts. Sample 9 contains mica, while samples 10 and 11 contain an appreciable amount of organic matter.

The soils represented by samples 6 to 11, inclusive, in addition to appreciable frost heaving, are likely to lose stability during thaws. In addition, these soils are capable of raising water in detrimental amounts through great heights during frost action.

Samples 12 and 13 were obtained from Maryland soils containing diatoms in appreciable amount. Soils of this character, especially when their shrinkage limits are equal to or greater than 50, are almost sure to produce pavement failure because of their high porosity.

FIGURE 63.—TYPICAL FROST BOIL CAUSED BY LACK OF STABILITY IN SOIL

TABLE 15.—*Group A–5 subgrades; constants of material passing the No. 40 sieve*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	67	0			84	69
2	58	0			36	62
3	43	17	41	1.3	48	36
4	50	11			[1] 64	45
5	47	0			22	28
6	37	8	30	1.5	[2] 51	32
7	36	9	28	1.5	[2] 40	31
8	45	18	37	1.4	54	37
9	37	11	33	1.5	41	35
10	40	8	33	1.3	36	37
11	43	9	37	1.3	38	41
12	81	19	60	0.9	89	87
13	119	42	94	0.7	94	107
14	49	0	42	1.2	34	44
15	34	12	26	1.5	25	30

[1] Tendency to waterlog. [2] Waterlogged.

TABLE 16.—*Mechanical analyses of Group A–5 subgrades*

Sample No.	Gravel particles larger than 2.0 millimeter	Coarse sand 2.0 to 0.25 millimeter	Fine sand 0.25 to 0.05 millimeter	Silt 0.05 to 0.005 millimeter	Clay below 0.005 millimeter	Colloids below 0.001 millimeter
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	0	1	7	86	[1]	[1]
2	0	0	10	78	12	8
3	0	0	9	74	17	7
4	0	0	1	6	93	26
5	0	0	8	78	14	4
6	0	2	5	16	77	17
7	0	0	14	27	59	40
8	0	1	6	32	61	8
9	0	1	5	60	24	2
10	3	6	23	57	14	0
11	3	6	27	60	7	0
12	0	0	29	42	29	12
13	0	0	40	28	32	9

[1] Flocculated.

With a specific gravity of 2.0, for instance, and a shrinkage limit equal to 50, these soils in their dried state have voids in amount equal to the volume occupied by the soil particles. Figure 64 shows a dried pat of sample 13 floating on the surface of water.

Samples 14 and 15 were taken from micaceous soils in Maryland. These samples are representative of the elastic subgrades which are productive of the early cracking in concrete pavements illustrated in Figure 15, A, Part I, of this report.

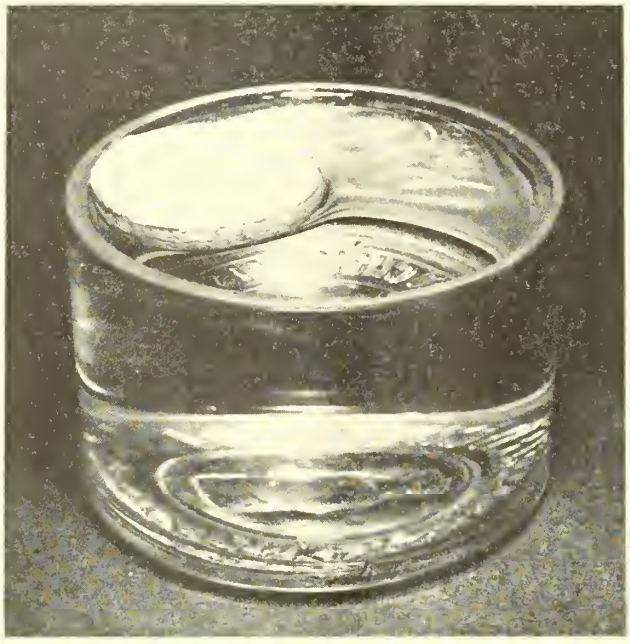


FIGURE 64.—DRIED PAT OF DIATOMACEOUS EARTH (SAMPLE 13, TABLE 15) FLOATING IN WATER AFTER BEING COATED WITH SHELLAC. APPARENT SPECIFIC GRAVITY LESS THAN ONE

A comparison of Tables 14 and 16 shows that it would be impossible to distinguish Group A–4 from Group A–5 subgrades on the basis of the mechanical analyses alone. The difference in the constants (Tables 13 and 15), however, indicates a difference in the characteristics of the soils. The higher liquid limits combined with the high shrinkage limits and field moisture equivalents differentiate those soils possessing elasticity (Group A–5) from those which are compressible (Group A–4).

Samples 4 and 7, according to the mechanical analyses, contain colloids in amount sufficient to produce high plasticity and shrinkage properties in appreciable amount, were the colloids active. These soils, however, according to their constants shown in Table 15, have low plasticity and negligible shrinkage properties and, consequently, should be grouped with the A–5 instead of the A–7 subgrades.

It is interesting to note that two soils (samples 7 and 9) which are likely to heave in appreciable amount have similar constants in spite of the fact that sample 7 contains 40 per cent of particles of colloidal size and 59 per cent of clay, whereas sample 9 contains but 2 per cent of colloidal particles and 24 per cent of clay. This is further evidence that grain size alone does not control the performance of subgrade soils.

Group A–6 subgrades.—The samples included in Tables 17 and 18 illustrate the constants and the grading characteristic of the nonclastic colloidal clay subgrades. Sample 1 was obtained from a colloidal clay soil which has proved troublesome because of sliding in a cut in Missouri, and sample 2 from a colloidal clay soil which has caused similar difficulties when used for fill material, in the same State. Samples 3 and 4 were composed of colloidal clays furnished by a survey of the soil existing under the Potomac River at Washington, D. C. These soils were considered unfit for use as hydraulic fill material. Sample 5 represents a very highly colloidal clay soil productive of landslides in Virginia. This soil contains about 30 per cent of material so fine that it remains in suspension for weeks. If located on an impervious undersoil, this soil when in a soft condition acts as a lubricant facilitating the sliding of the upper soil layers. (See Fig. 65.) Sample 6 was obtained from a colloidal soil from Texas, and sample 7, when occurring as subgrade, produced failure in a limerock base course being constructed on a road north of Gainesville, Fla.

TABLE 17.—*Group A–6 subgrades: constants of material passing the No. 40 sieve*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	51	31	13	1.9	[1] 56	34
2	55	36	13	1.9	[1] 55	36
3	58	40	12	1.9	[1] 45	28
4	57	40	12	1.9	[1] 45	28
5	132	101	11	1.9	[1] 178	33
6	75	49	11	2.0	[1] 90	30
7	50	32	18	1.7	35	30

[1] Waterlogged.

TABLE 18.—*Mechanical analyses of Group A–6 subgrades*

Sample No.	Gravel—particles larger than 2.0 millimeters	Coarse sand 2.0 to 0.25 millimeters	Fine sand 0.25 to 0.05 millimeters	Silt 0.05 to 0.005 millimeters	Clay below 0.005 millimeters	Colloids below 0.001 millimeters
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	0	0	1	52	40	15
2	0	0	11	57	32	16
3	0	19	30	11	50	24
4	16	25	12	30	43	27
5	0	3	3	6	88	78
6	0	3	20	29	48	24
7	0	28	32	17	23	10

The relatively large plasticity indices and the relatively small shrinkage limits and field moisture equivalents, combined with waterlogging in the centrifuge test, identify samples 1 to 6, inclusive, as containing clay highly active in character. This is emphasized by the mechanical analyses, which disclose that only one of the six samples contain more than 50 per cent of clay.

Sample 7, on the basis of its grading and its shrinkage limit would be classed an A–2 subgrade. Its volumetric change of 20.4 $((30-18)\times 1.7)$ and its plasticity index of 32 for a combined clay and silt content not larger than 40 definitely place this soil with the colloidal clays. The clay contained in this sample was similar in stickiness to chewing gum. This clay is the only subgrade material found thus far in the subgrade investigations which has an activity practically equal to that of bentonite. It is interesting to note that the plasticity index of this sample, which contains but 23 per cent of clay, is more than double that of lithopone, which contains 95 per cent of clay.

The presence of gluey colloids generally serves to protect the Group A–6 subgrades from detrimental frost heaving. These soils are likely to contain relatively large amounts of the unfreezable water productive of frost heave. The rate at which capillary moisture moves through them, however, is so slow that detrimental heave will probably not occur unless well-defined seepage planes furnish the necessary moisture at the required rate.

Group A–7 subgrades.—The samples contained in Tables 19 and 20 illustrate the grading and the constants characteristic of those clay subgrades which are likely to be elastic under certain prevalent conditions. Sample 1, furnished by F. V. Reagel, was taken from an expansive clay on which a concrete pavement in Missouri cracked in appreciable amount during the setting period of the concrete. Sample 2, furnished by W. C. McKnown, was obtained from an expansive clay in Kansas on which similar cracking occurred in a concrete pavement. Sample 3, furnished by W. D. Ross, was obtained from an expansive clay in Colorado on which similar cracking occurred in the pavement.

TABLE 19.—*Group A–7 subgrades; constants of material passing the No. 40 sieve*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	51	29	17	1.7	36	39
2	39	20	15	1.9	28	27
3	42	24	14	1.9	35	34
4	71	42	14	1.8	[1] 69	43
5	63	38	17	1.8	[1] 60	46
6	89	54	31	1.9	[2] 78	46
7	67	37	20	1.7	[2] 46	39
8	68	42	12	2.0	[1] 67	47
9	112	82	9	2.0	116	52
10	100	68	11	2.0	[1] 132	63
11	83	53	13	1.9	[1] 72	64

[1] Waterlogged. [2] Tendency to waterlog.

TABLE 20.—*Mechanical analyses of Group A–7 subgrades*

Sample No.	Gravel particles larger than 2.0 millimeters	Coarse sand 2.0 to 0.25 millimeters	Fine sand 0.25 to 0.05 millimeter	Silt 0.05 to 0.005 millimeter	Clay below 0.005 millimeter	Colloids below 0.001 millimeter
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	0	0	7	55	38	24
2	0	1	14	52	33	16
3	0	2	5	45	48	21
4	0	2	9	43	46	22
5	0	1	13	43	43	24
6	0	0	2	6	92	78
7	0	0	5	23	72	29
8	0	3	8	35	54	27
9	0	0	4	15	81	18
10	0	1	9	54	36	16
11	0	1	14	28	57	21

These three soils, it will be noted, are similar in several respects. They all contain approximately 50 per cent of silt; the plasticity indices of the three soils bear similar relationships to their liquid limits, being slightly less than those represented by curve 4; the centrifuge moisture equivalents of all three soils are relatively small, being approximately equal to those represented by curve 9; and the field moisture equivalents are either approximately equal to or greater than the centrifuge moisture equivalents. It will be recalled that the micaceous silts of the Group A–5 subgrade, on which cracking occurred during the early age of the concrete, also have field moisture equivalents generally larger than the centrifuge moisture equivalents.

Samples 4 and 5 were composed of gumbos from the Red River Valley, Minn., which do not suffer detrimental frost heave and which are stabilized when oil-treated and covered with granular material.

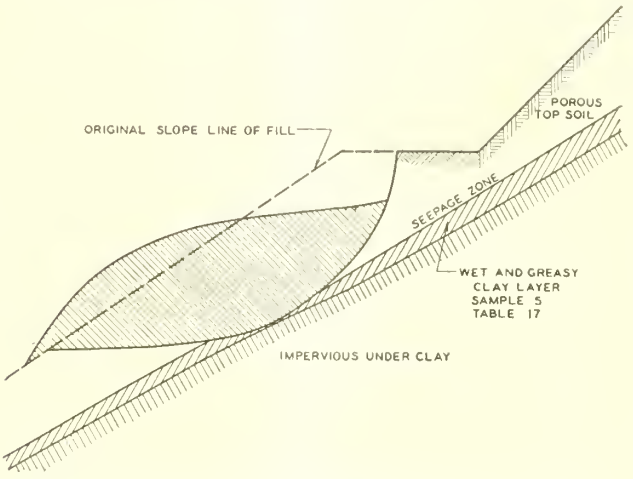

ORIGINAL SLOPE LINE OF FILL
POROUS TOP SOIL
SEEPAGE ZONE
WET AND GREASY CLAY LAYER SAMPLE 5 TABLE 17
IMPERVIOUS UNDER CLAY

FIGURE 65.—DIAGRAM ILLUSTRATING LANDSLIDE CAUSED BY PRESENCE OF HIGHLY COLLOIDAL CLAY SUCH AS SAMPLE 5, TABLE 17

Sample 6 was composed of kadox, which, according to laboratory experiments performed by Professor Taber (*24*), is productive of important frost heave. Except for low colloidal activity and high shrinkage limit this material seems to be a normal A–7 subgrade. The shrinkage limit of 31, however, is approximately equal to those of the Group A–5 silts of New Hampshire (samples 8 and 9, Table 15) in which detrimental heave due to frost is likely to occur.

The relative activity of the colloids in the two samples, 5 and 6, is disclosed when their constants are expressed as decimals of those characteristic of the average or statistical soils. Thus the plasticity index of sample 5 is 1.31 times that of an "average" soil containing 43 per cent of clay, and the plasticity index of sample 6 is only 0.76 times that of an "average" soil containing 92 per cent clay. Kadox, therefore, possesses both the relatively inactive colloids and the high porosity productive of frost heave.

Sample 7, obtained from a soil in Michigan similar to sample 6, has a relatively low plasticity index, only 0.69 times that of an "average" soil containing 72 per cent clay; and also a relatively high shrinkage limit, 1.82 times that possessed by the average soil referred to. This soil has been observed to have detrimental elasticity in dry weather. Increasing the moisture content of the soil, however, causes a decrease in its elasticity. This is true also for certain varieties of the micaceous silts.

This soil displays the properties indicative of frost heave. The current surveys, however, have not yet disclosed whether or not such heave occurs.

Sample 8 was obtained from a soil which became exceedingly troublesome because of sliding when used in a high fill in Missouri. The organic matter which causes this soil to be placed in the A–7 group has decomposed to such an extent that its presence is disclosed only by the relatively large field moisture equivalent.

Sample 9 represents a flocculated, highly colloidal soil from Mississippi. Only the absence of water-logging with a centrifuge moisture equivalent as large as 116 prevents this soil from being grouped with the A–6 subgrades. This soil, which contains both lime and gypsum, supports concrete slabs which have warped in detrimental amounts. Soil of this character can not be used efficiently in fills and in cuts it should be separated from concrete pavements by a good topsoil base course at least 2 feet thick.

Sample 10 was made up from a Texas black waxy soil. Its high field moisture equivalent causes it to be grouped with the A–7 subgrades. Sample 11 is another soil of the gumbo type which proved troublesome when used in fills in Arkansas.

FIGURE 66.—DRY LAND FILL CONSTRUCTED ON GROUP A–8 SUBGRADE IN VIRGINIA

Group A–8 subgrades.—Constants of representatives of the soft peats and mucks are contained in Table 21. The grading of these materials is not significant.

TABLE 21.—*Group A–8 subgrades; Constants of material passing the No. 40 sieve*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	61	22	33	1.4	[1] 62	51
2	62	26	38	1.4	[1] 56	47
3	59	19	31	1.4	68	51
4	265	0	141	0.5	263	265
5	445	0	187	0.3	395	440

[1] Waterlogged.

Samples 1 and 2 were obtained from Potomac River bottom muck which failed to support hydraulic fills without displacing laterally. The high shrinkage limits and high field moisture equivalents indicate the presence of partly decomposed organic matter. The relatively high plasticity indices and the water logging in the centrifuge test disclose the presence of clay or colloidal organic matter which in combination with the partly decomposed organic matter comprises muck. Sample 3 was obtained from Minnesota muck, which has a tendency to displace laterally when supporting fills. The presence of sand in this material may account for the absence of water logging in the centrifuge test. Sample 4 was composed of a peat which failed to support a dry land fill in Virginia (fig. 66) and sample 5 from a peat which has proved inadequate to support fills in Minnesota.

Samples 4 and 5 illustrate the very high water-absorptive properties characteristic of the peat soils which have not yet reached the colloidal state by decomposition. That these two soils have not yet reached this state is indicated by the fact that their plasticity indices are equal to zero and by the absence of water logging in the centrifuge test.

CONCLUSIONS SUMMARIZED

The foregoing discussion serves to emphasize that relatively few and comparatively simple laboratory tests may serve to identify fairly accurately the important characteristics of subgrade soils. Consequently these tests may serve to identify the dominating constituents composing the soils and to suggest the proper corrective measures to be used in pavement construction.

The following generalizations, based on the data given in the preceding pages, illustrate the service performed by the test constants in identifying the characteristics of subgrade soils.

Graded materials having centrifuge moisture equivalents greater than 15 have been found to be related to loss of subgrade stability in the presence of water.

Groups A–4, A–5, and A–7 subgrades having field moisture equivalents approximately equal to or greater than the centrifuge moisture equivalents have been found to be conducive to the cracking which occurs in pavements during the early life of the concrete.

Groups A–2, A–4, and A–5 subgrades having relatively high centrifuge moisture equivalents and Group A–7 subgrades having exceptionally high shrinkage limits seem to favor detrimental frost heave.

Group A–3 subgrades and groups A–6 and A–7 subgrades having relatively high plasticity indices are unlikely to heave under frost action.

Groups A–6 and A–7 subgrades are likely to shrink and expand in appreciable amount.

Group A–7 subgrades, having relatively high shrinkage limits, and Group A–5 subgrades, because of their elasticity, are likely to prove troublesome in the preparation of the subgrade.

The plastic varieties of the Group A–5 subgrades, with exceptionally low field moisture equivalents, generally have the properties required in good binders for sand-clay and topsoil roads.

The principal purpose of this report is to describe a method according to which soil research yielding profitable results may be performed. It should serve also to illustrate (*a*) the effort which may be required to interpret the test constants properly, (*b*) the character of the information which may be obtained by the intelligent use of the test constants, (*c*) the necessity for understanding the full significance of each constant, and (*d*) the impossibility of stating in simple terms the general procedures by means of which the constants of all soils may be readily interpreted.

It is expected that future invetigations will improve the procedure for making soil tests, and will increase the precision and facility with which soils may be identified by the use of test constants.

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Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.
Report of the Chief of the Bureau of Public Roads, 1929.
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Report of a Survey of Transportation on the State Highways of New Hampshire. (1927)

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio. (1928)

Report of a Survey of Transportation on the State Highways of Pennsylvania. (1928)

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

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Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

*Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID AND EMERGENCY ROAD CONSTRUCTION

AS OF

JUNE 30, 1931

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION						BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		ESTIMATED TOTAL COST	FEDERAL-AID ALLOTTED	EMERGENCY ADVANCE FUND	MILEAGE			ESTIMATED TOTAL COST	FEDERAL-AID ALLOTTED	EMERGENCY ADVANCE FUND	MILEAGE				
					INITIAL	STAGE*	TOTAL				INITIAL	STAGE*	TOTAL		
ALABAMA	2,116.8	$ 7,537,014.55	$ 3,671,288.23	$ 1,564,430.71	238.7	114.8	351.5	$ 83,312.53	$ 46,856.26	$ 34,691.58	1.4	2.7	4.1	$ 3,291,398.78	ALABAMA
ARIZONA	841.9	8,419,231.92	4,295,941.88	1,081,000.52	260.0	229.3	489.3	84,143.72	54,813.77	13,504.20	2.4	5.1	7.6	92,674.36	ARIZONA
ARKANSAS	1,737.7	9,139,186.13	4,354,305.88	1,373,157.00	235.6	102.1	337.7	1,491,745.71	713,622.88		23.7	29.8	53.8	114,578.53	ARKANSAS
CALIFORNIA	1,828.0	12,395,686.82	5,206,186.73	2,672,269.60	293.4	103.5	396.8	1,604,148.84	683,854.82	291,487.14	60.4		60.4	826,385.10	CALIFORNIA
COLORADO	1,343.3	5,781,305.18	3,115,215.83	1,319,888.20	206.2	89.0	295.2	328,875.27	184,752.46	51,871.71	15.8	21.4	37.3	1,958,342.74	COLORADO
CONNECTICUT	256.8	4,852,568.48	1,794,286.20	520,481.00	48.0		49.0	377,952.56	178,946.82		5.5		5.5	4,388.85	CONNECTICUT
DELAWARE	306.0	1,041,247.28	519,869.13	400,000.00	52.8		52.8	160,024.00	90,012.00		7.4		7.4	73,518.71	DELAWARE
FLORIDA	540.0	8,424,859.85	3,007,205.42	1,029,348.35	175.5		175.5	102,818.84	51,408.41	13,256.00	13.3		13.3	1,604,812.74	FLORIDA
GEORGIA	2,795.0	8,004,537.89	4,302,305.86	1,818,777.17	303.1	113.8	416.9	1,555,393.35	730,085.18	48,300.00	77.7	18.2	95.9	1,049,771.28	GEORGIA
IDAHO	1,281.9	4,085,734.81	2,297,771.02	968,534.92	271.2	32.9	304.1	987,824.38	574,435.01	137,770.21	58.3	55.9	114.2	395,029.73	IDAHO
ILLINOIS	2,257.7	25,922,268.77	11,785,892.09	2,376,700.00	789.3	41.1	810.4	8,257,424.68	3,778,078.65	1,012,300.00	271.8	4.7	278.8	279,005.33	ILLINOIS
INDIANA	1,579.0	9,209,958.89	4,536,328.72	1,030,313.38	281.3		281.3	3,030,777.18	1,453,338.00	353,212.70	129.5		129.5	755,222.13	INDIANA
IOWA	3,153.1	7,386,348.86	3,144,076.10	1,893,389.00	187.3	72.1	238.4	549,700.91	182,459.60	123,000.00		17.4	17.4	4,059.10	IOWA
KANSAS	3,148.8	8,883,373.58	4,247,356.40	1,535,966.37	505.6	55.8	561.4	1,604,248.75	885,140.83	175,947.54	113.0	46.2	159.2	386,029.70	KANSAS
KENTUCKY	1,519.0	7,438,643.91	3,259,483.02	1,195,190.78	284.4	131.1	415.5	880,912.21	428,092.98	238,600.00	60.5	19.8	80.8	183,559.55	KENTUCKY
LOUISIANA	1,418.8	8,422,172.79	4,005,813.00	1,137,927.00	245.9	18.3	265.2	120,355.82	58,515.30	10,000.00	5.1	.6	5.7	253,076.15	LOUISIANA
MAINE	603.2	4,337,387.40	1,642,395.30	637,799.00	101.0		101.0	1,084,871.78	470,583.82	43,000.00	18.9		18.9	912,261.07	MAINE
MARYLAND	707.0	1,662,189.40	759,448.81	582,772.48	47.9	4.2	52.1	405,165.40	202,577.68	88,339.88	17.7		17.7	81,940.51	MARYLAND
MASSACHUSETTS	724.2	9,210,169.31	2,275,526.32	802,509.00	78.8		78.9	1,467,104.81	563,800.20	268,000.00	17.0		17.0	1,597,158.35	MASSACHUSETTS
MICHIGAN	1,604.5	10,793,690.64	4,466,907.21	1,108,000.00	314.4	23.9	338.3	2,142,332.29	1,024,813.00	257,500.00	81.7	27.3	109.0	1,873,781.60	MICHIGAN
MINNESOTA	3,887.3	7,843,473.34	3,237,288.25	1,854,893.00	74.3	284.5	358.8	1,278,434.70	546,474.60	295,000.00	6.9	49.8	55.2	27,432.47	MINNESOTA
MISSISSIPPI	1,772.7	4,089,403.10	2,013,896.18	1,401,409.53	182.0	80.5	262.5	402,117.91	201,058.96	30,908.89	2.5	18.3	20.8	3,759,645.89	MISSISSIPPI
MISSOURI	2,658.5	9,829,670.52	3,905,276.83	2,264,272.94	248.8	63.0	311.8	1,879,887.13	848,926.77	213,472.79	81.2	25.8	87.0	30,847.49	MISSOURI
MONTANA	1,835.1	11,719,226.48	6,587,231.21	1,637,674.28	88.2	157.8	1,048.1	438,573.99	243,222.65	7,500.00	38.9	29.2	86.1	2,120,118.88	MONTANA
NEBRASKA	3,848.9	9,110,895.80	4,288,484.24	1,577,666.73	245.8	189.7	445.9	1,653,248.55	769,179.23	108,185.89	54.9	88.8	123.8	904,033.63	NEBRASKA
NEVADA	1,082.0	2,946,231.13	1,983,134.43	813,056.02	108.0	248.8	356.8	628,107.07	386,796.58	200,368.44	18.5	24.9	43.9	204,157.11	NEVADA
NEW HAMPSHIRE	382.8	1,343,033.48	515,208.60	270,000.00	26.1	1.0	27.1	485,120.12	127,744.38	130,000.00	3.4	7.3	10.7	94,801.88	NEW HAMPSHIRE
NEW JERSEY	555.2	9,209,545.44	1,794,968.33	1,103,770.59	74.7		74.7	144,716.24	3,420.00		.2		.2	1,177,568.27	NEW JERSEY
NEW MEXICO	1,927.9	6,820,755.52	4,117,268.84	1,259,460.48	251.5	145.5	397.0	107,277.63	67,700.35		.2		.2	98,785.38	NEW MEXICO
NEW YORK	2,684.0	39,415,270.33	13,701,573.50	3,714,566.00	680.8	7.0	687.8	9,191,200.00	2,428,525.00	230,000.00	115.9		115.9	15,421.72	NEW YORK
NORTH CAROLINA	1,870.6	8,585,115.87	3,126,179.57	1,628,473.82	213.1	35.4	248.5	1,023,035.04	503,233.00	233,527.78	29.0	36.4	85.4	1,608,127.82	NORTH CAROLINA
NORTH DAKOTA	4,363.2	4,377,846.70	2,233,117.38	961,855.89	453.3	457.4	910.7	877,431.89	438,715.84	118,900.00	90.4	184.9	285.3	1,087,248.83	NORTH DAKOTA
OHIO	2,563.6	15,298,336.93	4,780,217.25	1,830,069.07	231.1	21.9	262.7	5,068,860.88	1,964,858.48	1,188,488.93	85.9	23.4	119.3	1,236,443.47	OHIO
OKLAHOMA	1,981.3	7,764,820.32	3,678,809.87	1,601,789.48	298.4	117.7	416.1	1,078,318.17	578,499.44	308,814.37	38.0	13.9	52.9	8,011.81	OKLAHOMA
OREGON	1,267.7	7,584,617.07	4,175,255.21	1,012,430.03	246.8	60.3	337.1	728,600.13	382,647.87	292,423.05	49.1	5.4	67.5	442,218.87	OREGON
PENNSYLVANIA	2,664.4	12,715,867.38	5,194,853.58	2,713,838.94	251.7		251.7	2,700,355.73	1,295,683.77	787,079.41	96.4		96.4	1,151,378.69	PENNSYLVANIA
RHODE ISLAND	215.4	2,821,715.53	878,610.60	400,000.00	42.1		42.1	81,485.74	45,732.86		.1		.1	98,072.60	RHODE ISLAND
SOUTH CAROLINA	1,853.5	5,423,297.93	2,457,109.74	1,114,638.00	78.2	152.9	231.0							242,841.12	SOUTH CAROLINA
SOUTH DAKOTA	3,741.6	8,208,502.83	3,334,879.87	1,220,359.54	418.4	189.4	607.6	512,582.84	284,608.89	37,000.00	15.1	47.1	62.2	383,082.29	SOUTH DAKOTA
TENNESSEE	1,461.4	4,542,603.15	2,237,133.62	1,741,881.99	183.7	17.4	211.1	380,841.70	190,320.84		29.0		26.0	1,638,998.09	TENNESSEE
TEXAS	8,879.5	20,038,372.56	9,178,199.78	4,649,782.29	858.4	324.4	1,182.8	2,777,023.12	1,228,795.58	435,936.91	87.9	51.2	138.8	3,256,872.24	TEXAS
UTAH	1,041.5	2,204,486.97	1,284,752.41	608,159.04	133.8	38.9	172.9	819,142.92	483,403.55	221,262.73	31.2	92.8	124.0	524,243.32	UTAH
VERMONT	301.6	1,291,829.51	529,907.38	365,064.10	30.5	4.7	35.2	99,418.48	25,868.93	10,785.86	2.3		2.3	52,345.06	VERMONT
VIRGINIA	1,824.7	9,193,780.18	2,610,806.22	1,467,181.72	260.9	48.2	307.1	497,760.77	248,238.51	9,200.00	18.4	.8	18.0	578,394.71	VIRGINIA
WASHINGTON	1,001.4	5,833,892.87	2,590,959.02	1,188,780.48	175.7	26.8	202.5	774,880.71	328,442.43	61,308.65	12.7	17.1	29.9	893,899.52	WASHINGTON
WEST VIRGINIA	772.8	5,158,102.15	2,037,787.87	993,827.31	129.8	12.5	142.4	628,633.87	274,083.33	135,830.00	9.1	12.9	22.0	334,335.88	WEST VIRGINIA
WISCONSIN	2,414.8	5,618,261.82	3,783,296.04	1,805,000.00	228.8	74.5	303.4	1,690,746.84	552,719.04	182,211.21	48.9	17.8	88.6	82,080.01	WISCONSIN
WYOMING	1,629.0	4,584,178.27	2,794,003.06	899,674.61	351.0	242.4	593.4	311,033.98	155,515.00	116,800.00	10.7	43.9	54.9	152,707.71	WYOMING
HAWAII	47.9	1,429,014.40	560,743.37	394,628.83	41.1		41.1	82,324.99	22,889.00		1.6		1.6	1,679,875.17	HAWAII
TOTALS	88,713.1	387,396,991.92	172,587,244.08	87,308,514.17	12,306.8	4,174.0	19,479.8	60,185,458.60	28,885,946.86	8,470,225.84	1,945.9	1,033.3	2,978.9	39,838,888.04	TOTALS

(*) THE TERM STAGE CONSTRUCTION REFERS TO ADDITIONAL WORK DONE ON PROJECTS PREVIOUSLY IMPROVED WITH FEDERAL-AID. IN GENERAL, SUCH ADDITIONAL WORK CONSISTS OF THE CONSTRUCTION OF A SURFACE OF HIGHER TYPE THAN WAS PROVIDED IN THE INITIAL IMPROVEMENT.

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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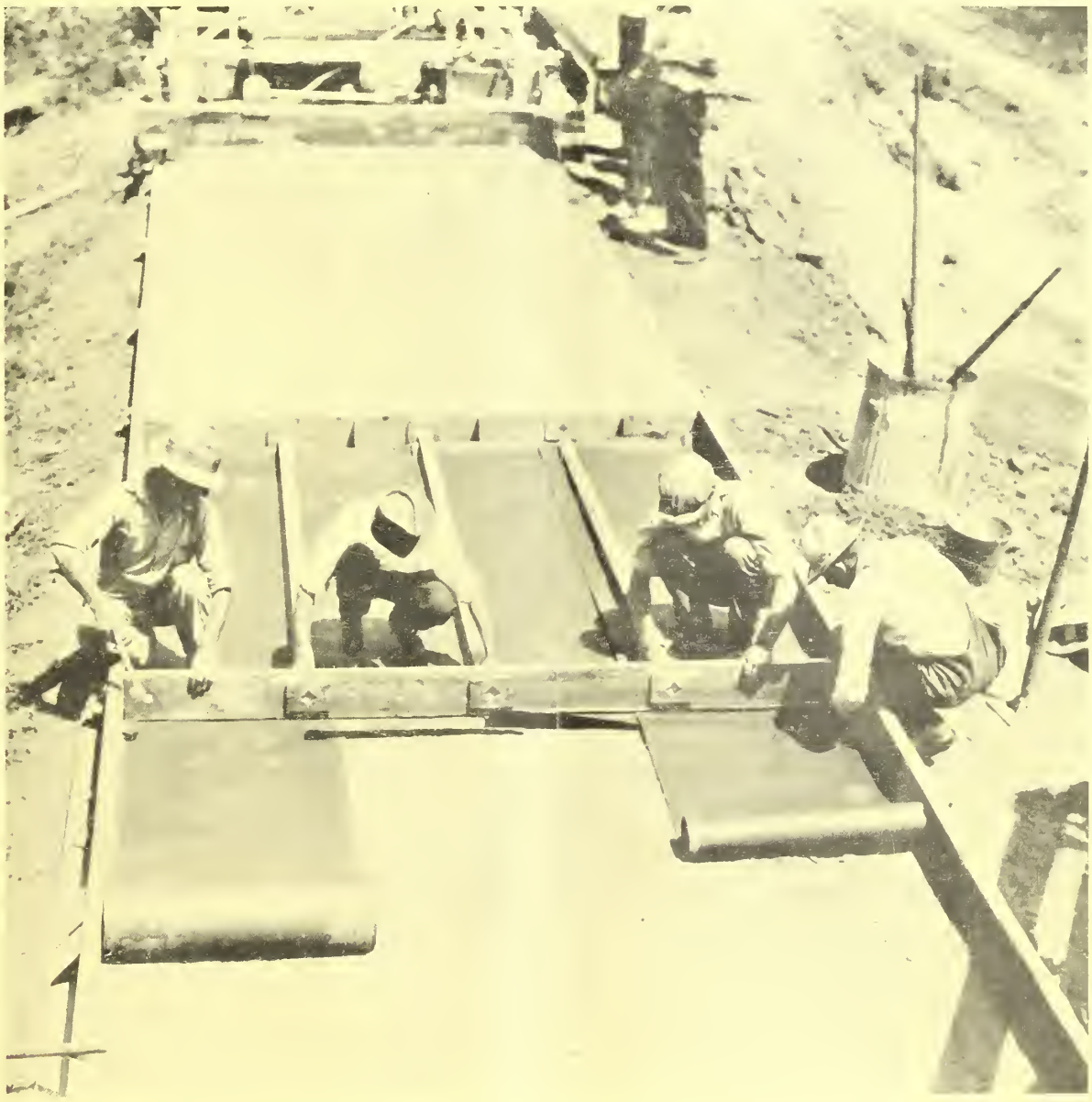

PREPARING TEST SLABS FOR STUDIES OF PAVING CONCRETE

For sale by the Superintendent of Documents, Washington, D. C. See page 2 of cover for prices

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

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STUDIES OF PAVING CONCRETE[1]

By F. H. JACKSON, Senior Engineer of Tests, and W. F. KELLERMANN, Associate Materials Engineer, U. S. Bureau of Public Roads

THE tests which are reported in this paper were undertaken primarily for the purpose of determining the effect of variations in the quantity of coarse aggregate upon the strength, density, and other properties of concrete pavement slabs which had been placed and finished in accordance with normal field practice. Experience in the use of multiple sizes of coarse aggregate in concrete pavement work in North Carolina and other States during the past few years has indicated the possibility of increasing the quantity of coarse aggregate per unit of volume of concrete beyond the limit which had previously been considered good practice and in this way producing a denser as well as a more economical mixture, provided only that the uniformity of the grading of the coarse aggregate and therefore its void content was controlled rigidly by handling and measuring it in separate sizes.

It was realized at the outset that an investigation of this sort, involving as it did that most elusive property of concrete which we call "workability," could not be performed satisfactorily in the laboratory. In spite of strenuous efforts on the part of many investigators, no satisfactory laboratory test for workability has as yet been developed. Furthermore, this whole matter of workability is tied up so intimately with methods of handling and finishing used on the job that it is impossible to set up any laboratory standard which will give more than comparative results. In other words, a concrete which by some laboratory standard may be rated as "unworkable" may be quite workable under certain job conditions. For this investigation the percentage of visible honeycomb in the concrete, as revealed by a careful examination of the slabs, all of which were constructed in accordance with standard field practice, together with the uniformity of strength as determined by testing beams taken directly from the pavement, has been used to measure the uniformity and therefore the workability of the concrete. On this basis any concrete mixture which can be so handled as to produce a uniform homogeneous slab without unduly raising labor costs or reducing efficiency in operation is "workable" concrete, and any concrete which can not be so handled is not workable, in spite of any rating which it may receive by some arbitrary test.

Most of the information which we have regarding the properties of concrete at the present time has been obtained in the laboratory on small-sized beams and cylinders. While these data are of great value in helping us to understand some of the fundamental relationships governing the quality of concrete, as, for instance, the relation between water-cement ratio and strength, we must realize that it is the finished structure with which we are primarily concerned and that we are not in a position to make the best use of laboratory test data until we know to just what extent tests on molded specimens measure the quality of the concrete in the structure. This investigation furnished an excellent opportunity to study these relationships, both for crushing strength, through the use of molded cylinders compared with cores drilled from the pavement, and for transverse strength, through the use of molded beams compared with large beams taken from the pavement.

PROJECT DESCRIBED

In order to develop information along these lines, the Bureau of Public Roads during the summer of 1929 constructed an experimental concrete pavement 9 feet wide and approximately one-half mile long, using standard construction methods and appliances throughout, except that provision was made for creating planes of weakness to permit the removal of the pavement in sections for test purposes. The pavement was built at the Arlington Experiment Station of the Department of Agriculture at Arlington, Va., on the right of way of an abandoned electric line. This provided a graded, well drained, level subgrade which proved ideal for the purpose.

A total of 265 sections, each 9 feet in length, was constructed. The program called for the construction of six sections per day, three sections of each of two proportions, using a given type and gradation of coarse aggregate and method of finishing. In each group of three the water content was varied so as to produce a variable consistency ranging from the driest mix which it appeared possible to place without undue effort on the part of the finishers to a consistency approximating a 2 to 3 inch slump. No effort was made to produce wet consistencies such as have sometimes been used in the past, because it was felt that the dangers of overwet concrete are sufficiently well known and require no demonstration. On the other hand, the water-cement ratio method of proportioning, the adoption of which is being strongly urged, encourages the use of dry concrete, so that it seemed desirable to study mixtures of this sort rather than the wet consistencies which are recognized as undesirable by everyone.

MATERIALS AND PROPORTIONS

In order to cover adequately the question of type of coarse aggregate as it affected the workability of the concrete and therefore the limiting quantity which might be used with safety, a siliceous limestone having a rather sharp angular fracture and a bank gravel containing some crushed fragments were used in the tests. In addition, a limited number of sections were laid with blast furnace slag as coarse aggregate.

The cement was a standard brand Portland, meeting all requirements of the American Society for Testing Materials. It was shipped by car direct from the mill to the site of the work and all of the cement came from one bin. The results of physical tests of this cement are given in Table 1.

The fine aggregate consisted of sand from the Potomac River, having a fineness modulus of 2.65. Average test data are given in Table 1. It was realized, of course, that the grading and other characteristics of the sand used in these experiments would have a marked effect upon the amount of coarse aggregate which could

[1] The bureau desires to express its appreciation of the courtesy extended by the following companies in loaning the equipment indicated in each case for use during the construction of the test pavement:
National Equipment Co., 27E paver.
Blaw-Knox Co., 2-compartment bin with weighing batcher and steel road forms.
Heltzel Steel Form & Iron Co., 2-compartment bin with weighing batcher and teel road forms.
Lakewood Engineering Co., Lakewood combination single screed and tamper finishing machine.
A. W. French Co., Ord double-screed finishing machine.
Duquesne Slag Products Co., crushed slag.

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TABLE 1.—*Physical properties of cement and fine aggregate*

1. Portland cement:	
Fineness, percentage retained on 200-mesh sieve	14. 7
Time of set (Gillmore)—	
Initial	3 hours, 8 minutes.
Final	6 hours, 3 minutes.
Steam test for soundness	Satisfactory.
Normal consisteney, per cent	22. 6
Tensile strength (pounds per square inch, 1:3 Ottawa sand mortar)—	
At 7 days	315
At 28 days	400
(Results are average of 14 samples tested.)	
2. Fine aggregate:	
Sieve analysis [1]—	
Total retained on ¼-inch sereen, per cent	1
Total retained on No. 10 sieve, per cent	14
Total retained on No. 20 sieve, per cent	32
Total retained on No. 30 sieve, per cent	45
Total retained on No. 40 sieve, per cent	69
Total retained on No. 50 sieve, per cent	86
Total retained on No. 80 sieve, per cent	94
Total retained on No. 100 sieve, per cent	96
Total retained on No. 200 sieve, per cent	97
Silt and elay, per cent	2. 8
Apparent speeifie gravity	2. 65
Weight per eubie foot (dry-rodded), pounds	102
Absorption (Rea's method), per cent	0. 7
Voids, per cent	38
Organie matter (eolor test)	Satisfactory.
Strength ratio—	
7 days	96
28 days	113
Deseription: Sand eonsists essentially of angular quartz grains, eontaining some ehert, feldspar, and miea.	

[1] Sieve analysis is average of 96 samples tested.

TABLE 2.—*Physical properties of coarse aggregates*

Type of aggregate	Specific gravity	Absorption	Wear	Weight per cubic foot (dry rodded)	Voids
		Per cent	*Per cent*	*Pounds*	*Per cent*
Gravel, grading A	2. 67	0. 27	14. 3	108	35
Gravel, grading B	2. 64	. 43		105	36
Stone, grading A	2. 72	. 12	[1] 4. 4	102	40
Stone, grading B	2. 72	. 12		101	41
Slag	2. 47		12. 2	87	44

[1] Not standard test; made with crushed rock.

TABLE 3.—*Gradings of coarse aggregate*

Aggregate	Grading	Per cent by weight passing round-opening screens			
		2½-inch	1¼-inch	¾-inch	¼-inch
Gravel	A	100	50	25	0
Do	B		100	60	0
Stone	A	100	50	25	0
Do	B	100	37	10	0
Slag	A	100	50	25	0

be used. It is well known that in general the finer the sand the higher the percentage of coarse aggregate by volume which may be employed without exceeding the limits of workability. However, it was obviously impossible to study any great range in fine aggregate gradations in a program such as this. It was decided, therefore, to use a single sand of average gradation with varying percentages of coarse aggregate, beginning at a value sufficiently low to insure a well-oversanded mix for the particular type of sand selected and increasing the amount of coarse aggregate by definite steps until a distinctly undersanded mix, as judged by laboratory standards, had been obtained.

The results of the physical tests of the three coarse aggregates are given in Table 2. The limestone was obtained from Martinsburg, W. Va.; the gravel from Fredericksburg, Va.; and the slag from Birdsboro, Pa. Each aggregate was ordered shipped to the job in three separate sizes, as follows: ¼-inch to ¾-inch, ¾-inch to 1¼-inch, 1¼-inch to 2½-inch. The separated sizes were combined by weight to give the combined aggregate gradations indicated in Table 3.

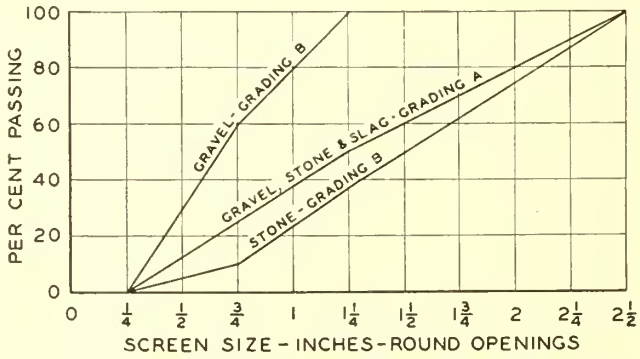


FIGURE 1.—GRADINGS OF COARSE AGGREGATES AS DESIGNED

It will be noted that in the case of the gravel and the crushed stone two gradings were employed. Grading A is the so-called straight-line grading, showing an even distribution of sizes from the maximum down to ¼-inch. Grading B in the case of the gravel ranged in size from ¼-inch to 1¼-inch, with a surplus in the finer sizes. This grading was used because of the economic importance of this type of gravel in certain sections of the country. Grading B for the crushed stone showed a deficiency in the finer sizes and was used to simulate a condition often met with in practice whenever a demand exists for these finer sizes for bituminous work, resulting in a tendency to rob the concrete aggregate of these sizes. The sieve analysis curves for the combined gradings are shown in Figure 1.

MORTAR-VOIDS ANALYSIS MADE

In the preparation of this report an analysis was made of the base mix, 1: 2: 3½, used in these tests according to the Talbot-Richart mortar-voids theory.[2] Results of mortar-voids tests on the sand used, which were made by Professor Richart, of the University of Illinois, form the basis of this analysis. According to the mortar-voids theory, for materials similar in quality, the strength of concrete is a function of the ratio of the volume of voids to the volume of cement in the mix. The theory also holds that, for the type of plastic mix ordinarily employed in construction, the void characteristics of the mortar constituent may be used in investigating the probable strength of the concrete. This is possible because, for such mixes, the volume of mortar is greater than the volume of voids in the coarse aggregate, and the voids in the concrete may therefore be considered as made up of the sum of the air and water voids in the mortar. The results of the mortar-voids analysis are omitted from the present report for lack of space.

In the method of designing concrete described by Talbot and Richart the maximum quantity of coarse aggregate to use is determined by means of the ratio $\frac{b}{b_0}$ in which b is the absolute volume of coarse aggregate in a unit volume of concrete and b_0 is the absolute volume of coarse aggregate in a unit volume of coarse

[2] Bulletin 137, University of Illinois Engineering Experiment Station, 1923.

aggregate. This ratio expresses directly the bulk volume (absolute volume plus voids) of coarse aggregate which is present in a unit volume of concrete. Since the voids in a given volume of coarse aggregate are always increased by the addition of mortar, the volume of resulting concrete will always be larger than the bulk volume of coarse aggregate used, and the ratio $\frac{b}{b_0}$ will always be less than 1. In designing concrete mixes the practical application of this theory involves the determination of b for a given b_0 on the basis of an assumed value for the ratio, the magnitude of which will vary with the several conditions affecting workability, such as type of coarse aggregate, methods of placing and finishing, etc. A number of State highway departments are using this ratio in designing their paving mixtures, the value being based, in most cases, upon field practice. For this reason the values for $\frac{b}{b_0}$ corresponding to the various arbitrary mixes selected for this study have been calculated and are given in Table 11. The value of the ratio for each mix has been calculated on the basis of the average amount of water used in the various sections in which the mix was employed.

CONSTRUCTION METHODS OUTLINED

Batching materials.—Although the basic proportions were determined by dry-rodded volumes, all of the materials for each batch, with the exception of water, were weighed. The aggregates were weighed in regulation batchers; the cement was weighed in the original sacks on platform scales. In order to handle the coarse aggregate in three sizes, two 2-bin batcher plants were used. One of these handled two sizes of coarse aggregate, while the other handled the third size and the sand. The batcher plants were of different make, one being provided with an automatic dial for indicating the weight and the other with a beam and rider. The latter was equipped also with an automatic tell-tale for indicating over or under weight. The weighing hoppers on both batchers were so arranged that any excess material could be conveniently removed. A view of the batcher plant layout is shown in Figure 2.

As the bins were filled, sieve analyses were made of the separate sizes of coarse aggregate and these data later used in determining the weights of each size necessary to give the theoretical grading required. It was ascertained from these analyses that certain of the sizes contained appreciable quantities of material which passed the smaller screen designating that particular size. However, the percentage of fines in each size did not vary to any extent from day to day, so that, a correction having been established for the undersize material, it was not necessary to change it often.

Filling of the sand hopper was not begun until the morning on which the material was to be used. In this way variations in moisture content within the hopper were eliminated to a large extent. Moisture determinations were made as the hopper was filled, and these percentages were used in calculating the weight of dry sand required for each batch. As the sand was delivered into the skip of the mixer, additional samples were taken and these samples sent to the laboratory for mechanical analysis and a check on the moisture content.

Hauling material from batching plant to mixer.—The materials were conveyed from the batching plant by dump trucks, each truck hauling one batch. The trucks first received two separate sizes of coarse aggregate from one batcher, then pulled under the second batcher for the third size and the sand, and then passed by the cement house where the cement was loaded in sacks and partial sacks. Immediately prior to the approach of the trucks to the mixer these sack were opened and the cement spread over the aggregate. Canvas spread over the damp sand in the trucks prevented loss of moisture during the interval between loading and dumping into the skip of the mixer.

Construction of test slabs.—The concrete was laid as a pavement 9 feet in width, 7 inches thick, and on a flat subgrade which had been very carefully prepared. In order to insure uniform subgrade conditions, single-ply felt tar paper was placed on the subgrade prior to laying

FIGURE 2.—PROPORTIONING PLANT USED IN CONSTRUCTION OF TEST SECTIONS

the concrete. Transverse headers 9 feet apart separated the sections, each of which therefore had a volume of 47 cubic feet, or just a little less than the volume of two standard-size batches of a 27E paver. As the concrete was actually to be taken up and tested, three 2-inch by 2-inch wooden separators were placed between the headers in such a manner as to create longitudianl planes of weakness and to divide each 9-foot by 9-foot section into four slabs each 27 inches wide by 9 feet in length. These 2-inch by 2-inch strips were placed at the center of the 7-inch section, and, being dry when placed, absorbed sufficient moisture from the concrete to cause swelling, with subsequent longitudinal cracking in the concrete. A view of the subgrade showing the separators in place is shown on the cover. A completed section appears in the background. Before the concrete had set, a line of ¾-inch holes was punched across each section 2 feet from each end. Immediately prior to testing, plugs and feathers were inserted in these holes and the 2-foot end strips broken off. Thus there were obtained from each section four slabs approximately 27 inches wide by 5 feet in length by 7 inches in depth. These slabs were tested in flexure by a method described later.

Mixing concrete.—For mixing the concrete, a new 27E paver was employed. It was equipped with a tank for measuring water, which was filled by gravity from an auxiliary tank mounted on top of the mixer. The auxiliary tank had a greater capacity than the measuring tank and provided a means of supplying the mixer with water while hose connections to the water line were being changed. The use of the auxiliary tank also relieved the measuring tank of all pipe-line pressure, thereby making water measurements more positive.

All batches were mixed for one minute, the mixer being provided with a timing device and indicator bell.

The mixer ran between the side forms, and the first batch for each section was deposited in such a manner as to fill one-half of the section for its entire length. After the second batch was deposited the mixer was moved forward and the next section prepared by placing the tar paper, headers, and separators while the section just placed was being finished. Figure 3 shows the first batch being dumped. Figure 4 shows the finishing machine beginning the operation of spreading the concrete. At the beginning of each day's run

FIGURE 3.—DUMPING FIRST BATCH

a preliminary one-half batch was run and discarded, in order to place the inside of the drum in the same condition for the first batch as for all subsequent batches. Mixing operations for each day were continuous, so that it was not necessary to repeat this operation.

Determining weights for each batch.—The quantity of concrete required for each section plus that necessary for the control specimens amounted to about 51 cubic feet. It was decided to make two 27 cubic-foot batches for each test section. This was accomplished by computing the quantities of the constituent materials on the basis of absolute volume for each mix. The only change necessary was an adjustment in water content at the mixer in order to have three consistencies for each proportion. This did not necessitate any change in the weights at the batching plant as the water-cement ratio used in the computations was that required for the driest consistency. It did result, however, in slightly higher yields for the wetter consistencies.

Finishing.—Two finishing machines were employed for finishing the concrete, a combination single screed and tamper, and a double-screed machine. In the first round of tests the tamper was used with the single-screed machine while in the second round the screed was used without the tamper. The double screed was used in each round of tests. In the tables and charts the single screed with tamper is referred to as type A, the single screed without tamper as type B, and the double screed as type C. Both machines were equipped with belts so that no hand belting was necessary. Figures 5, 6, and 7 show typical views of the finishing operations as performed by type A, the single screed and tamper, and type C, the double-screed machine. Figure 5 illustrates, for the two types of machine, the typical appearance of the concrete before the first

TABLE 4.—*Schedule of test sections giving type of coarse aggregate and finishing machine used on each section*

ROUND 1

Section Nos.	Coarse aggregate	Finishing machine
1–18	Gravel, grading A	Type A [1]
19–33	Gravel, grading B	Do.
34–51	Gravel, grading A	Type C. [2]
52–68	Gravel, grading B	Do.
69–83	Crushed stone, grading A	Type A.
84–97	Crushed stone, grading B	Do.
98–112	Crushed stone, grading A	Type C.
113–127	Crushed stone, grading B	Do.

ROUND 2

128–145	Gravel, grading A	Type C.
146–160	Gravel, grading B	Do.
161–178	Gravel, grading A	Type B. [3]
179–193	Gravel, grading B	Do.
194–208	Crushed stone, grading A	Do.
209–223	Crushed stone, grading B	Do.
224–238	Crushed stone, grading A	Type C.
239–253	Crushed stone, grading B	Do.
254–265	Crushed slag, grading A	Do.

[1] Single screed with tamper.
[2] Double screed.
[3] Single screed without tamper.

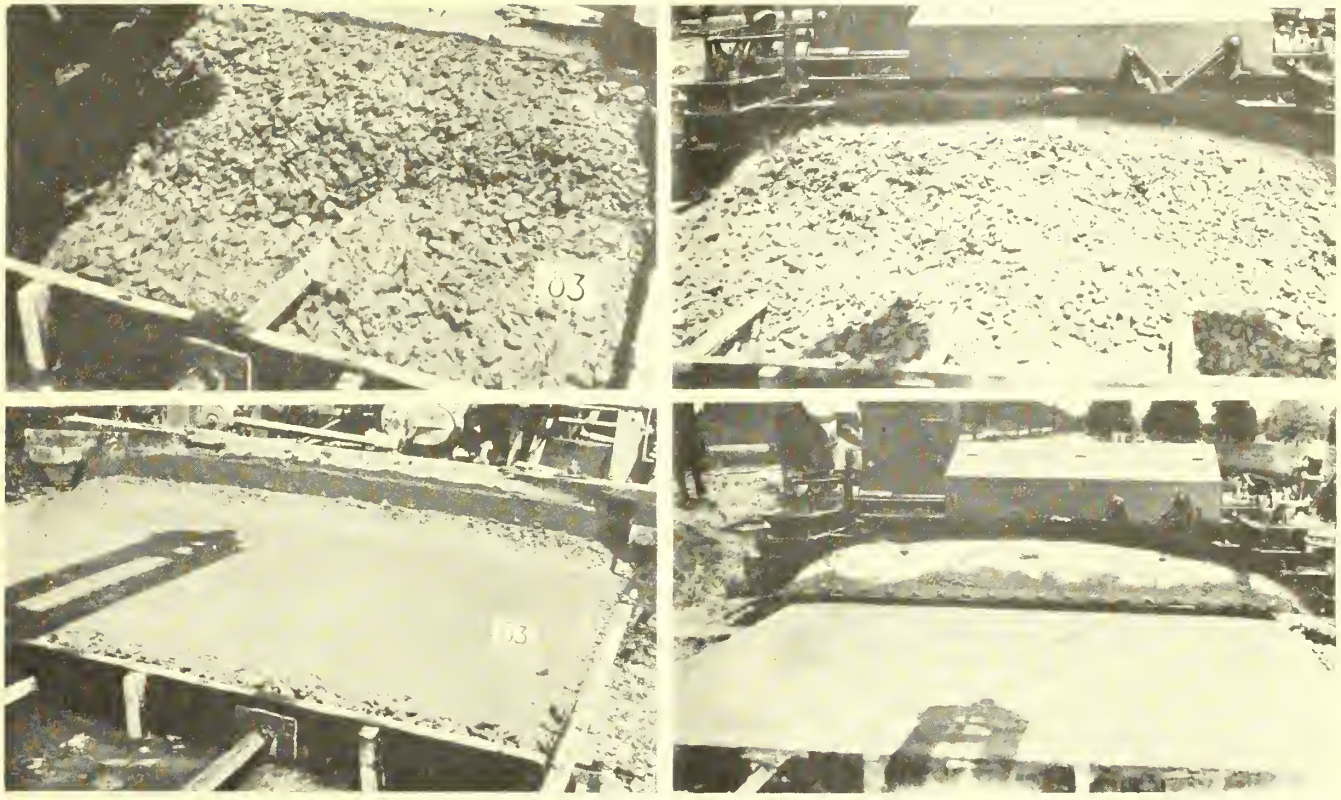
FIGURE 4.—FINISHER SPREADING CONCRETE

passage of the finisher and the appearance after the passage of the screeds. Figure 6 shows the final appearance of the section after belting. A finish of this character was obtained on practically all sections. Figure 7 illustrates the finishing operations in detail.

The machines were operated according to the schedule given in Table 4.

In general, the number of passes of the finishing machine was limited to three, with the idea that this would be about the economic limit from the standpoint of production management on an actual job. However, no more finishing was done, in any case, than was necessary to secure a satisfactory surface finish. Complete notes were taken giving the number of passes of the finishing machine over each test section, the number of times the concrete was tamped in the case of type A machine and the amount of hand work, if any, necessary. These notes are tabulated in Table 12.

Curing.—Wet burlap was applied to the pavement as soon after laying as the surface would permit. This burlap was kept wet during the day on which the pavement was laid. On the following morning the burlap was removed and a layer of earth about 2 inches deep was applied. This earth was kept wet for 10 days, after which it was left on the slab but not wet down. Fourteen days prior to testing the earth was wet down and kept wet until the individual sections were tested.



Type A finisher in operation on 1:2:4¾ crushed stone concrete, grading A, slump 2 inches

Type C finisher in operation on 1:2:4½ crushed stone concrete, grading A, slump 1½ inches

FIGURE 5.—FINISHING OPERATIONS ON SECTIONS 83 AND 107

CONTROL TESTS CONDUCTED FOR EACH TEST SECTION

Four beams and four cylinders were made as control specimens for each test section. The beam molds were 7 by 7 by 30 inches in size, four to a gang, and were made of wood. The cylinder molds were 6 by 12 inches, were of steel, and rested on machined base plates. These plates in turn rested on a wooden platform which also served as a base for the beam molds. The platform was placed on a level bearing about 4 feet from the roadway, thus permitting sufficient space for the finishing machine and workmen on the test section. Very little water was lost from within the specimens. This end was accomplished by using two thicknesses of felt paper between the beam molds and base and heavy grease between the cylinder molds and plates. Wet burlap was placed over the test specimens to prevent loss of water by evaporation.

The concrete for the control specimens was taken in the following manner: After the first batch was dumped from the mixer a sample weighing about 350 pounds was shoveled into two water-tight pans, in equal parts. A second sample similarly divided was taken after the second batch was dumped, placed in the pans together with the samples from the first batch, and the two samples in each pan thoroughly mixed with shovels. Great care was exercised to obtain representative samples. Slump and flow tests were then made, beams and cylinders fabricated, and the excess concrete returned to the test section. In making the beam specimens the following procedure was followed: The molds were filled in two layers, each layer being rodded 75 times with a ⅝-inch steel rod, bullet-shaped on the end, and then spaded on the sides and ends. After

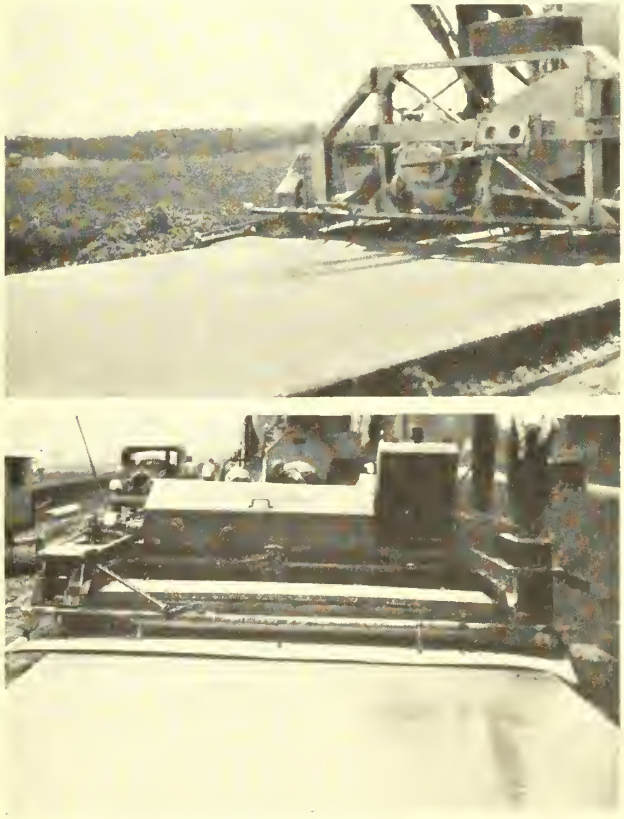
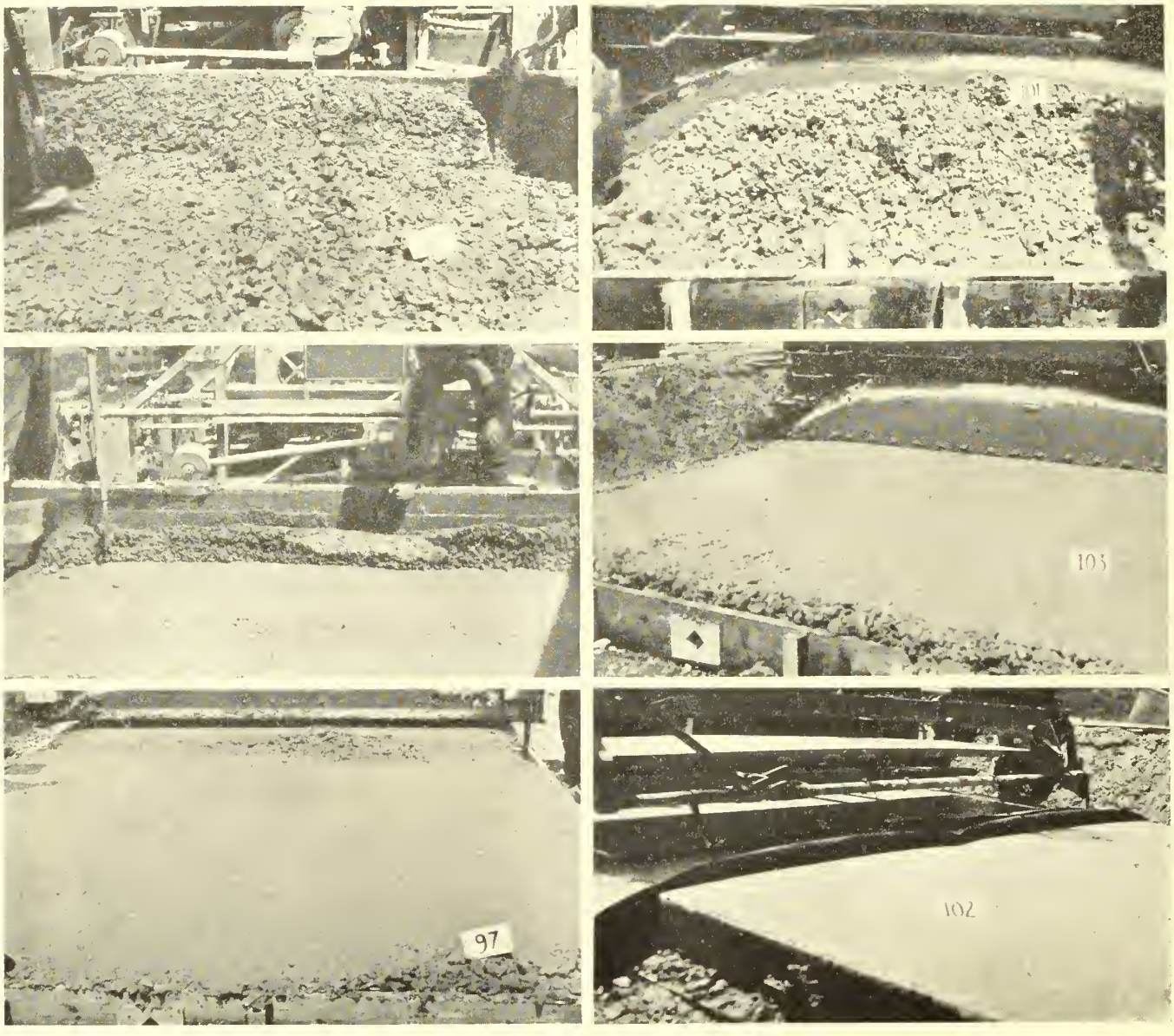


FIGURE 6.—TYPICAL APPEARANCE OF FINISHED SECTIONS. TOP, TYPE A FINISHER; BOTTOM, TYPE C FINISHER



Type A finisher

Type C finisher

FIGURE 7.—TYPICAL VIEWS OF FINISHERS IN OPERATION

the second layer was completed the top was struck off and finished with a wooden float. The cylinders were made in three layers in accordance with the practice of the American Society for Testing Materials and finished with a wooden float. After about 22 hours the specimens were removed from the molds and placed by the side of the test section. They were covered with wet earth at the time the slab was covered.

The slump tests were made in accordance with the tentative method of test for consistency of Portland cement concrete of the American Society for Testing Materials. The flow tests were made on a 30-inch flow table, the test consisting of 15 drops of ½-inch in 10 seconds, as described in the publication, "A. S. T. M. serial designation C 39–27, Standard Method of Making Compression Tests of Concrete." The values are given in Table 5.

Two of the four control beams were tested at 28 days and the other two at 9 months. The corresponding cylinders were tested at 28 days and 10 months. All control specimens were immersed in water for 24 hours before testing to insure uniform moisture content.

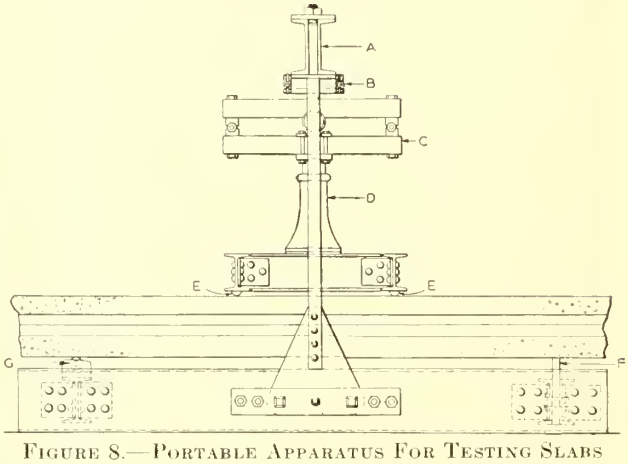


FIGURE 8.—PORTABLE APPARATUS FOR TESTING SLABS

In the case of the 28-day cylinders the specimens were removed from the subgrade at an age of 11 days and brought to the laboratory to be capped. They were

then returned to wet-earth storage until 24 hours before testing. The 10-month cylinders were handled in the same manner except that they were removed from the subgrade one month before testing instead of 17 days as in the case of the 28-day specimens.

Flexure tests were made on a 100,000-pound universal testing machine, the idle crosshead speed being 0.05 inch per minute. The beams were tested with the bottom as molded in tension, span 27 inches, load applied at the third points.

FIGURE 9.—TESTING SLAB FOR FLEXURAL STRENGTH

SPECIAL APPARATUS CONSTRUCTED FOR TESTING SLABS

Because of the large number of flexure tests to be made on the 27 by 60 inch slabs it was quite impracticable to make the tests in the laboratory, and field testing was mandatory. The importance of the data given by the flexure tests was such that accuracy commensurate with that obtained in the laboratory was essential. For these reasons special consideration was given to the items of accuracy and mobility in the design of the apparatus for making these tests.

The general design was such that the machines could be quickly disassembled into units which could be picked up and moved by hand; and, yet with a designed load capacity of 20,000 pounds, the total load on the specimen could be determined within about 25 pounds.

Figure 8 shows a side elevation of one of the testing machines. It consists of a structural steel base frame which supports a transverse rocker bearing F and roller G on which the slab to be tested is supported. At the center of each side member of the base frame a vertical tension member is attached by two easily removable pins. Passing transversely across the slab at the upper end of these two tension members is a steel beam A, which takes the reaction when a load is applied on the slab. The load is applied by means of a ball-bearing ratchet jack D and is distributed to the third points of the span by a structural frame E. For determining the magnitude of the applied load a pair of heat-treated steel beams C is inserted between the head of the jack and the reaction beam A. The combined deflection of the two beams is measured with a micrometer dial reading in ten-thousandths of an inch. The load-deflection characteristics of the beam combination having been determined in advance by calibration, it becomes a simple matter to translate the dial readings into terms of total applied load. This method of measuring load has been used previously (see PUBLIC ROADS, vol. 7, No. 2, April, 1926) and has proved to be very satisfactory. To assure an even bearing of the knife edge located in the center of the upper beam against the reaction a cylindrical bearing block B was provided.

In order to take care of the number of tests which had to be made, two complete machines and an extra base frame were built. This permitted the load-applying equipment to be shifted, after the slab failed, to an unbroken specimen set up for test on the extra base, greatly expediting the work.

TESTS OF PAVEMENT SLABS DESCRIBED

As previously explained, the installation of planes of weakness during construction had caused the formation of three equally spaced longitudinal cracks in each 9-foot slab, so that there were available for testing four slabs each 27 inches wide and 9 feet long. Each of these slabs was reduced in length to 5 feet just prior to testing by breaking off a section 2 feet in length from each end. These end sections were discarded entirely because it was felt that, because of the resistance offered by the header boards to free movement of the

concrete, the material in the end sections might not be truly representative.

Since the test slabs weighed approximately 1,000 pounds each, it was necessary to use a special rig for mounting them on the testing machine. A number of A frames sufficiently long to span the entire test section and equipped with trolley and chain hoist were employed for this purpose. Four clamps slipped under the edges of the slabs near the corners and connected with the chain hoist by means of chains provided with hooks proved entirely adequate for handling these slabs.

A uniform bearing across the entire width of slab at E, F, and G was obtained by using plaster of Paris and thin metal strips. Rubber strips were substituted for plaster of Paris at E whenever the top of the slab was smooth enough to permit it.

The specimens were tested by applying the load by hand as shown in Figure 9. The magnitude of the load at failure was determined by measuring the deflection of the calibrated beams with the micrometer dial, also shown in the figure. It was necessary to note the maximum reading of the dial, as the hand returned to zero the instant the slab broke. It was not difficult to catch this reading, since the hand on the dial would move very slowly as the maximum loading was approached. In general, two observers read the dial independently and checked their readings after failure.

CORES DRILLED FROM PAVEMENT

The breaking of the four test slabs of each section produced 8 half slabs. Cores were drilled from 2 of these 8 pieces, and the cores were tested for absorption at an age of approximately 9½ months. The specimens were boiled in water for 5 hours and then dried in an oven at a temperature of about 160° C. to constant weight. Additional cores were drilled from the broken slabs and tested in compression at 15 months. These cores were immersed in water for 24 hours before testing.

In order to provide specimens for durability tests, additional cores from a number of typical sections were drilled for each mix and are being subjected to alternate freezing and thawing at this time. A report covering this phase of the investigation will be issued later.

TEST RESULTS ANALYZED IN DETAIL

The data resulting from these tests are presented both in tabular and in graphic form. The results are shown in detail section by section and are also summarized in various ways in order to bring out certain relationships which appear to have been established.

The discussion is developed along the following lines: Certain general relationships which appear to exist between the strength and uniformity of the concrete and each of the variables which have been introduced are first considered. This preliminary analysis is followed by a discussion of the detail results for the purpose of ascertaining to what extent the results for individual sections deviate from the average trends and to determine if possible the reasons for such deviations. The following variables are discussed in the order named, the three coarse aggregates, gravel, crushed stone, and slag being considered separately in each case:

1. Relation between strength of concrete and mix.
2. Relation between strength of concrete and variations in cement factor resulting from change in mix.
3. Relation between strength of concrete and variations in water-cement ratio resulting from change in mix.
4. Effect of type of finishing machine.
5. Effect of grading of coarse aggregate.
6. Effect of honeycomb in slabs on modulus of rupture for each type of finishing machine.
7. Relation between honeycomb in slabs and mix.
8. Relation between honeycomb in slabs and consistency.
9. Uniformity of concrete.
10. Absorption tests.

The detailed data of the tests arranged section by section in the order of laying are shown in Tables 5 and 6. These tables contain all of the data from which the tables and charts showing average values were prepared, with the exception of the material relative to uniformity, honeycombing, and absorption.

TABLE 5.—*Proportions and consistency of concrete*

ROUND 1, GRAVEL AGGREGATE, GRADING A, TYPE A FINISHER

Section No.	Date laid (1929)	Proportions	Water-cement ratio	Slump in inches	Flow	Theoretical cement factor in sacks per cubic yard
1	July 10	1:2:3½	0.85	2		5.56
2	July 11	1:2:3½	.82	1¼	125	5.59
3	July 10	1:2:3½	.83	¼		5.58
4	do	1:2:4	.90	1¾		5.16
5	do	1:2:4	.87	1		5.18
6	do	1:2:4	.93	1¾		5.12
7	July 11	1:2:4½	.90	1¼	125	4.86
8	do	1:2:4½	.86	1	122	4.89
9	do	1:2:4½	.99	2½	144	4.78
10	do	1:2:4¾	1.00	4	155	4.64
11	do	1:2:4¾	.95	3	150	4.68
12	July 12	1:2:4¾	.87	1	125	4.74
13	do	1:2:5	.92	1¾	130	4.57
14	do	1:2:5	1.01	5	160	4.50
15	do	1:2:5	.96	3	148	4.53
16	do	1:2:5¼	.96	2¾	138	4.42
17	do	1:2:5¼	.91	1¾	132	4.46
18	do	1:2:5¼	1.03	4¾	164	4.37

ROUND 1, GRAVEL AGGREGATE, GRADING B, TYPE A FINISHER

19	July 15	1:2:3½	0.89	1	124	5.54
20	do	1:2:3½	.93	2¼	144	5.50
21	do	1:2:3½	.90	1¼	127	5.53
22	do	1:2:4	.85	1¼	127	5.24
23	do	1:2:4	.92	2	132	5.17
24	do	1:2:4	.96	2½	136	5.14
25	July 16	1:2:4½	.94	1¼	117	4.86
26	do	1:2:4½	1.01	1¾	138	4.80
27	do	1:2:4½	1.05	3	135	4.76
28	do	1:2:4¾	1.05	2	139	4.63
29	do	1:2:4¾	.99	2	139	4.68
30	do	1:2:4¾	.96	1¼	119	4.70
31	July 17	1:2:5	1.04	2	138	4.52
32	do	1:2:5	.92	¾	120	4.61
33	do	1:2:5	.96	1	116	4.58

ROUND 1, GRAVEL AGGREGATE, GRADING A, TYPE C FINISHER

34	July 18	1:2:3½	0.86	1	123	5.54
35	do	1:2:3½	.90	2½	138	5.50
36	do	1:2:3½	.87	1¾	134	5.53
37	do	1:2:4	.86	¾	128	5.19
38	do	1:2:4	.93	1	123	5.12
39	do	1:2:4	.94	1¾	130	5.11
40	July 19	1:2:4½	.91	1½	120	4.85
41	do	1:2:4½	.88	1	122	4.87
42	do	1:2:4½	.97	1¾	130	4.80
43	do	1:2:4¾	.90	1	118	4.71
44	do	1:2:4¾	.98	1¾	125	4.64
45	do	1:2:4¾	1.03	1½	144	4.61
46	July 22	1:2:5	.93	1¼	115	4.56
47	do	1:2:5	1.00	2	122	4.51
48	do	1:2:5	1.03	3	144	4.49
49	do	1:2:5¼	.93	1¾	130	4.44
50	do	1:2:5¼	.90	¼	111	4.46
51	do	1:2:5¼	1.03	2½	140	4.37

ROUND 1, GRAVEL AGGREGATE, GRADING B, TYPE C FINISHER

52	July 23	1:2:3½	0.84	¾	125	5.60
53	do	1:2:3½	.89	1½	135	5.54
54	do	1:2:3½	.93	2¾	143	5.50
55	do	1:2:4	.88	1¼	127	5.21
56	do	1:2:4	.95	2½	140	5.14
57	do	1:2:4	.99	3	142	5.10
58	July 24	1:2:4½	.93	1½	128	4.86
59	do	1:2:4½	.98	¾	126	4.82
60	do	1:2:4½	1.04	2	142	4.77
61	do	1:2:4¾	.96	1¼	122	4.70
62	do	1:2:4¾	1.03	2¼	134	4.65

TABLE 5.—*Proportions and consisteney of concrete*—Continued

ROUND 1, GRAVEL AGGREGATE, GRADING B, TYPE C FINISHER—Continued

Section No.	Date laid (1929)	Proportions	Water-cement ratio	Slump in inches	Flow	Theoretical cement factor in sacks per cubic yard
63	July 24	1:2:4¾	1.10	3	146	4.59
64	July 25	1:2:5	1.02	¾	122	4.53
65	do	1:2:5	1.10	2	132	4.47
66	do	1:2:5	1.13	3	148	4.45
67	do	1:2:5¼	1.05	2	131	4.39
68	do	1:2:5¼	.98	0	115	4.44

ROUND 1, STONE AGGREGATE, GRADING A, TYPE A FINISHER

69	July 26	1:2:3½	0.84	2½	133	5.77
70	do	1:2:3½	.81	1¼	125	5.81
71	do	1:2:3½	.75	¼	110	5.88
72	do	1:2:4	.84	1½	127	5.42
73	do	1:2:4	.90	2¼	132	5.36
74	do	1:2:4	.80	½	112	5.47
75	July 29	1:2:4¼	.88	¾	120	5.22
76	do	1:2:4¼	.94	2¼	146	5.16
77	do	1:2:4¼	.85	¼	117	5.25
78	do	1:2:4½	.85	½	120	5.10
79	do	1:2:4½	.90	1½	123	5.06
80	do	1:2:4½	.94	2¼	134	5.02
81	July 30	1:2:4¾	.87	½	115	4.95
82	do	1:2:4¾	.91	1½	120	4.91
83	do	1:2:4¾	.99	2	133	4.84

ROUND 1, STONE AGGREGATE, GRADING B, TYPE A FINISHER

84	July 30	1:2:3½	0.85	2	138	5.81
85	do	1:2:3½	.81	½	116	5.86
86	do	1:2:3½	.89	2¼	132	5.76
87	July 31	1:2:4	.87	1	124	5.43
88	do	1:2:4	.91	1¾	138	5.39
89	do	1:2:4	.94	2½	146	5.36
90	do	1:2:4¼	.89	1½	130	5.25
91	do	1:2:4¼	.94	2	128	5.20
92	do	1:2:4¼	.97	2¾	144	5.17
93	Aug. 1	1:2:4½	.89	0	120	5.10
94	do	1:2:4½	.95	1¼	132	5.05
95	do	1:2:4½	.99	1	150	5.00
96	do	1:2:4¾	.92	1¼	134	4.95
97	do	1:2:4¾	1.05	2	158	4.83

ROUND 1, STONE AGGREGATE, GRADING A, TYPE C FINISHER

98	Aug. 2	1:2:3½	0.83	1	118	5.78
99	do	1:2:3½	.88	1¾	140	5.72
100	do	1:2:3½	.81	½	118	5.81
101	do	1:2:4	.85	¾	120	5.41
102	do	1:2:4	.90	1¾	135	5.36
103	do	1:2:4	.95	2½	145	5.30
104	Aug. 5	1:2:4¼	.87	1¼	122	5.23
105	do	1:2:4¼	.94	3	144	5.16
106	do	1:2:4¼	.87	1½	122	5.23
107	do	1:2:4½	.91	1½	128	5.05
108	do	1:2:4½	.95	2¾	130	5.00
109	do	1:2:4½	.99	2½	142	4.97
110	Aug. 6	1:2:4¾	.92	½	112	4.90
111	do	1:2:4¾	.99	1½	125	4.84
112	do	1:2:4¾	1.04	2½	150	4.80

ROUND 1, STONE AGGREGATE, GRADING B, TYPE C FINISHER

113	Aug. 6	1:2:3½	0.85	1¾	127	5.81
114	do	1:2:3½	.89	½	136	5.76
115	do	1:2:3½	.92	3	155	5.72
116	Aug. 7	1:2:4	.91	2	140	5.39
117	do	1:2:4	.87	1¾	144	5.43
118	do	1:2:4	.94	1¾	142	5.36
119	do	1:2:4¼	.97	3	156	5.17
120	do	1:2:4¼	.91	2	146	5.23
121	do	1:2:4¼	.85	1	118	5.29
122	Aug. 8	1:2:4½	.89	½	117	5.10
123	do	1:2:4½	.95	1½	138	5.05
124	do	1:2:4½	.98	3	160	5.02
125	do	1:2:4¾	.96	2½	147	4.91
126	do	1:2:4¾	.94	2¾	147	4.93
127	do	1:2:4¾	1.03	3	180	4.85

ROUND 2, GRAVEL AGGREGATE, GRADING A, TYPE C FINISHER

128	Aug. 9	1:2:3½	0.85	1½	127	5.57
129	do	1:2:3½	.83	1½	122	5.59
130	do	1:2:3½	.90	2	130	5.51
131	do	1:2:4	.95	2½	152	5.10
132	do	1:2:4	.89	2½	121	5.16
133	do	1:2:4	.86	1	124	5.19
134	Aug. 14	1:2:4½	.88	1½	132	4.87

ROUND 2, GRAVEL AGGREGATE, GRADING A, TYPE C FINISHER—Continued

Section No.	Date laid (1929)	Proportions	Water-cement ratio	Slump in inches	Flow	Theoretical cement factor in sacks per cubic yard
135	Aug. 14	1:2:4½	.98	2	148	4.79
136	do	1:2:4½	.92	2¾	147	4.84
137	do	1:2:4¾	.91	1½	123	4.70
138	do	1:2:4¾	.96	2	138	4.66
139	do	1:2:4¾	1.02	2½	151	4.62
140	Aug. 15	1:2:5	.93	1½	118	4.56
141	do	1:2:5	.99	2	144	4.52
142	do	1:2:5	1.02	2¾	130	4.49
143	do	1:2:5¼	.94	1½	130	4.43
144	do	1:2:5¼	.98	1¼	119	4.40
145	do	1:2:5¼	1.03	2¼	142	4.37

ROUND 2, GRAVEL AGGREGATE, GRADING B, TYPE C FINISHER

146	Aug. 16	1:2:3½	0.84	1	122	5.60
147	do	1:2:3½	.89	1½	132	5.54
148	do	1:2:3½	.94	2¾	148	5.49
149	do	1:2:4	.87	½	117	5.22
150	do	1:2:4	.92	1¾	125	5.17
151	do	1:2:4	.97	2	129	5.12
152	Aug. 19	1:2:4½	1.03	2	133	4.78
153	do	1:2:4½	.98	1½	123	4.83
154	do	1:2:4½	.99	1	124	4.81
155	do	1:2:4¾	.94	1	111	4.72
156	do	1:2:4¾	1.00	2	127	4.67
157	do	1:2:4¾	1.05	2¾	130	4.63
158	Aug. 20	1:2:5	1.01	1½	115	4.54
159	do	1:2:5	1.07	1½	129	4.49
160	do	1:2:5	1.13	2¾	148	4.45

ROUND 2, GRAVEL AGGREGATE, GRADING A, TYPE B FINISHER

161	Aug. 21	1:2:3½	0.83	¾	124	5.58
162	do	1:2:3½	.88	1	120	5.52
163	do	1:2:3½	.93	2	136	5.47
164	do	1:2:4	.90	1½	124	5.15
165	do	1:2:4	.95	2¼	132	5.10
166	do	1:2:4	.88	1	127	5.17
167	Aug. 22	1:2:4½	.88	1	115	4.87
168	do	1:2:4½	.92	1½	118	4.84
169	do	1:2:4½	.98	1¾	124	4.79
170	do	1:2:4¾	.91	1¼	123	4.70
171	do	1:2:4¾	.96	1¾	124	4.66
172	do	1:2:4¾	1.02	2	133	4.62
173	Aug. 23	1:2:5	1.07	3	152	4.46
174	do	1:2:5	.99	1	141	4.52
175	do	1:2:5	.92	1	124	4.57
176	do	1:2:5¼	1.03	2¼	138	4.37
177	do	1:2:5¼	.98	1	130	4.40
178	do	1:2:5¼	.94	1½	125	4.43

ROUND 2, GRAVEL AGGREGATE, GRADING B, TYPE B FINISHER

179	Aug. 26	1:2:3½	0.89	2	133	5.54
180	do	1:2:3½	.85	1	126	5.59
181	do	1:2:3½	.94	2¼	139	5.49
182	do	1:2:4	.97	2½	147	5.12
183	do	1:2:4	.92	1½	133	5.17
184	do	1:2:4	.87	¾	113	5.22
185	Aug. 27	1:2:4½	1.04	3½	138	4.77
186	do	1:2:4½	.98	1¾	137	4.82
187	do	1:2:4½	.94	1¾	124	4.86
188	do	1:2:4¾	1.05	2	145	4.63
189	do	1:2:4¾	1.00	1¾	139	4.67
190	do	1:2:4¾	.94	1¼	125	4.72
191	Aug. 28	1:2:5	1.13	2½	146	4.45
192	do	1:2:5	1.07	1	163	4.49
193	do	1:2:5	1.01	1	134	4.54

ROUND 2, STONE AGGREGATE, GRADING A, TYPE B FINISHER

194	Aug. 29	1:2:3½	0.88	1½	134	5.72
195	do	1:2:3½	.92	2¼	156	5.67
196	do	1:2:3½	.97	4	155	5.61
197	do	1:2:4	.95	2¼	146	5.30
198	do	1:2:4	.90	1¼	131	5.36
199	do	1:2:4	1.00	3	157	5.25
200	Aug. 30	1:2:4¼	.94	2½	138	5.16
201	do	1:2:4¼	.99	3	157	5.11
202	do	1:2:4¼	.89	2	128	5.21
203	do	1:2:4½	.99	2¾	169	4.97
204	do	1:2:4½	.94	2¼	147	5.02
205	do	1:2:4½	.89	1¾	137	5.07
206	Sept. 3	1:2:4¾	1.04	3¼	143	4.80
207	do	1:2:4¾	.99	2¼	146	4.84
208	do	1:2:4¾	.93	1½	131	4.89

TABLE 5.—*Proportions and consistency of concrete*—Continued

ROUND 2, STONE AGGREGATE, GRADING B, TYPE B FINISHER

Section No.	Date laid (1929)	Proportions	Water-cement ratio	Slump in inches	Flow	Theoretical cement factor in sacks per cubic yard
209	Sept. 3	1:2:3½	0.92	2¼	150	5.72
210	do	1:2:3½	.87	2	135	5.78
211	do	1:2:3½	.84	1½	134	5.82
212	Sept. 4	1:2:4	.96	2	144	5.34
213	do	1:2:4	.92	1¾	142	5.38
214	do	1:2:4	.87	¾	137	5.43
215	do	1:2:4¼	1.00	2½	160	5.14
216	do	1:2:4¼	.95	1¾	138	5.19
217	do	1:2:4¼	.90	1¾	142	5.24
218	Sept. 5	1:2:4½	1.05	3¼	166	4.95
219	do	1:2:4½	.99	2¼	162	5.01
220	do	1:2:4½	.91	2¼	142	5.08
221	do	1:2:4¾	1.08	4	170	4.80
222	do	1:2:4¾	.94	1¾	141	4.93
223	do	1:2:4¾	.90	2	135	4.96

ROUND 2, STONE AGGREGATE, GRADING A, TYPE C FINISHER

224	Sept. 6	1:2:3½	0.97	4	161	5.61
225	do	1:2:3½	.92	2¼	156	5.67
226	do	1:2:3½	.85	1½	130	5.76
227	do	1:2:4	1.00	3¾	150	5.25
228	do	1:2:4	.92	2¼	143	5.34
229	do	1:2:4	.87	1½	130	5.39
230	Sept. 9	1:2:4¼	.89	1¼	132	5.21
231	do	1:2:4¼	.94	2	143	5.16
232	do	1:2:4¼	.99	3	145	5.11
233	do	1:2:4½	.92	1¼	135	5.04
234	do	1:2:4½	.97	2¼	133	4.99
235	do	1:2:4½	1.02	2½	138	4.95
236	Sept. 10	1:2:4¾	.93	2	131	4.89
237	do	1:2:4¾	.99	2¼	145	4.84
238	do	1:2:4¾	1.04	3½	166	4.80

ROUND 2, STONE AGGREGATE, GRADING B, TYPE C FINISHER

Section No.	Date laid (1929)	Proportions	Water-cement ratio	Slump in inches	Flow	Theoretical cement factor in sacks per cubic yard
239	Sept. 10	1:2:3½	0.84	2	142	5.82
240	do	1:2:3½	.87	2¼	147	5.78
241	do	1:2:3½	.92	2¾	158	5.72
242	Sept. 11	1:2:4	.87	1½	123	5.43
243	do	1:2:4	.92	2¼	134	5.38
244	do	1:2:4	.97	1¾	154	5.33
245	do	1:2:4¼	.89	1½	123	5.25
246	do	1:2:4¼	.94	2	130	5.20
247	do	1:2:4¼	.84	1	120	5.30
248	Sept. 12	1:2:4½	.92	¾	128	5.07
249	do	1:2:4½	.88	2	133	5.11
250	do	1:2:4½	.99	3¼	145	5.01
251	do	1:2:4¾	.90	2¼	128	4.96
252	do	1:2:4¾	.96	2	138	4.91
253	do	1:2:4¾	.98	2¼	145	4.89

ROUND 1, SLAG AGGREGATE, GRADING A, TYPE C FINISHER

254	Sept. 13	1:2:3½	0.87	1½	126	5.91
255	do	1:2:3½	.92	1¾	128	5.84
256	do	1:2:3½	.83	¾	115	5.96
257	do	1:2:4	.87	½	108	5.57
258	do	1:2:4	.92	2	124	5.51
259	do	1:2:4	.99	2¼	138	5.43
260	Sept. 16	1:2:4¼	.94	1½	125	5.34
261	do	1:2:4¼	.99	1¾	114	5.28
262	do	1:2:4¼	.92	1	118	5.36
263	do	1:2:4½	.98	1¾	130	5.15
264	do	1:2:4½	1.03	2½	128	5.10
265	do	1:2:4½	.95	1¼	130	5.18

TABLE 6.—*Results of strength tests*

ROUND 1, GRAVEL AGGREGATE, GRADING A, TYPE A FINISHER

Section No.	Proportions	Modulus of rupture in pounds per square inch: Beams 28 days	Beams 9 months	Slabs, 9 months	Compressive strength in pounds per square inch: Cylinders 28 days	Cylinders 10 months	Cores, 15 months
1	1:2:3½	434	479	542	3,085	3,980	5,320
2	1:2:3½	468	581	573	2,940	3,900	4,780
3	1:2:3½	508	582	504	3,300	4,460	4,920
4	1:2:4	427	515	554	2,900	3,940	4,660
5	1:2:4	392	525	532	3,260	4,480	5,120
6	1:2:4	434	471	520	2,475	3,460	3,950
7	1:2:4½	478	501	529	2,785	3,910	4,700
8	1:2:4½	489	549	575	2,940	3,950	5,400
9	1:2:4½	442	491	544	2,410	3,600	4,290
10	1:2:4¾	404	453	530	2,250	3,160	4,240
11	1:2:4¾	448	490	564	2,495	3,260	4,440
12	1:2:4¾	460	434	497	3,145	3,680	4,420
13	1:2:5	452	471	557	3,130	3,880	4,320
14	1:2:5	454	458	529	2,330	3,130	4,270
15	1:2:5	484	455	562	2,690	3,240	4,140
16	1:2:5¼	448	409	500	2,340	3,210	3,920
17	1:2:5¼	456	526	453	2,975	3,720	4,420
18	1:2:5¼	388	460	535	2,100	3,040	3,900

ROUND 1, GRAVEL AGGREGATE, GRADING B, TYPE A FINISHER

19	1:2:3½	446	624	609	3,310	5,000	4,900
20	1:2:3½	491	661	590	3,515	4,520	5,100
21	1:2:3½	527	602	580	3,590	4,530	5,260
22	1:2:4	518	614	542	3,580	4,580	5,120
23	1:2:4	556	593	559	3,730	4,670	5,080
24	1:2:4	457	653	561	2,970	4,420	4,360
25	1:2:4½	510	605	466	3,660	4,450	4,660
26	1:2:4½	489	585	593	3,200	4,160	4,960
27	1:2:4½	470	554	566	3,210	3,800	3,940
28	1:2:4¾	458	519	541	2,540	3,740	4,660
29	1:2:4¾	440	561	503	3,415	4,420	4,150
30	1:2:4¾	491	506	468	3,500	4,410	4,620
31	1:2:5	399	478	519	2,470	3,040	4,140
32	1:2:5	469	660	430	3,440	4,750	5,170
33	1:2:5	462	642	447	3,305	4,410	4,770

ROUND 1, GRAVEL AGGREGATE, GRADING A, TYPE C FINISHER

Section No.	Proportions	Modulus of rupture in pounds per square inch: Beams 28 days	Beams 9 months	Slabs, 9 months	Compressive strength in pounds per square inch: Cylinders 28 days	Cylinders 10 months	Cores, 15 months
34	1:2:3½	482	662	583	3,360	4,600	5,170
35	1:2:3½	534	679	547	3,315	4,540	4,820
36	1:2:3½	544	701	583	3,360	4,310	5,020
37	1:2:4	510	682	594	3,535	4,010	4,880
38	1:2:4	523	670	511	3,165	4,080	4,740
39	1:2:4	494	690	508	3,260	3,920	4,840
40	1:2:4½	522	500	452	3,280	3,980	4,440
41	1:2:4½	441	485	518	3,430	4,740	4,450
42	1:2:4½	426	525	540	2,755	4,040	4,440
43	1:2:4¾	441	573	537	3,215	3,600	4,600
44	1:2:4¾	448	488	535	3,185	3,820	4,360
45	1:2:4¾	461	487	543	2,750	3,700	4,180
46	1:2:5	411	479	537	3,095	3,540	4,140
47	1:2:5	405	516	483	2,570	3,340	4,750
48	1:2:5	396	538	520	2,430	2,800	4,210
49	1:2:5¼	393	509	464	2,885	3,370	4,100
50	1:2:5¼	386	562	432	3,305	4,380	4,580
51	1:2:5¼	397	428	565	2,490	3,470	4,420

ROUND 1, GRAVEL AGGREGATE, GRADING B, TYPE C FINISHER

52	1:2:3½	555	569	447	3,985	4,300	4,310
53	1:2:3½	588	633	571	3,760	4,280	4,410
54	1:2:3½	491	553	572	3,480	4,110	4,640
55	1:2:4	427	566	585	3,525	4,190	4,460
56	1:2:4	519	513	555	3,325	3,980	4,430
57	1:2:4	495	545	579	3,270	3,900	4,360
58	1:2:4½	490	579	451	3,645	4,370	4,230
59	1:2:4½	503	562	516	3,335	4,280	4,490
60	1:2:4½	470	511	551	3,145	3,740	4,470
61	1:2:4¾	496	599	537	3,715	4,200	4,450
62	1:2:4¾	407	530	536	2,970	3,750	4,220
63	1:2:4¾	469	495	547	2,785	3,410	3,680
64	1:2:5	371	483	413	3,075	3,810	4,390
65	1:2:5	475	544	477	2,805	3,450	3,850
66	1:2:5	474	590	535	2,645	3,190	3,760
67	1:2:5¼	496	524	501	3,110	3,880	3,990
68	1:2:5¼	436	593	448	3,175	4,110	3,980

TABLE 6.—*Results of strength tests*—Continued

ROUND 1, STONE AGGREGATE, GRADING A, TYPE A FINISHER

Section No.	Proportions	Modulus of rupture in pounds per square inch: Beams 28 days	Beams 9 months	Slabs, 9 months	Compressive strength in pounds per square inch: Cylinders 28 days	Cylinders 10 months	Cores, 15 months
69	1:2:3½	611	716	732	3,430	4,390	5,410
70	1:2:3½	614	810	725	3,830	4,600	5,190
71	1:2:3½	721	822	690	3,990	5,020	5,200
72	1:2:4	702	735	687	3,795	4,430	5,010
73	1:2:4	632	751	697	3,720	4,500	4,860
74	1:2:4	705	793	632	3,890	4,710	5,190
75	1:2:4¼	637	791	725	3,380	3,980	4,920
76	1:2:4¼	580	788	660	2,685	3,500	4,500
77	1:2:4¼	643	860	708	3,575	4,130	5,040
78	1:2:4½	691	880	670	3,650	4,080	5,340
79	1:2:4½	647	777	700	3,335	4,140	4,780
80	1:2:4½	526	738	711	3,020	3,970	4,540
81	1:2:4¾	506	754	657	3,070	3,720	4,580
82	1:2:4¾	474	786	681	2,725	4,050	4,910
83	1:2:4¾	510	752	678	2,420	3,080	4,050

ROUND 1, STONE AGGREGATE, GRADING B, TYPE A FINISHER

84	1:2:3½	541	640	742	3,385	3,930	4,760
85	1:2:3½	556	713	653	3,480	4,670	4,480
86	1:2:3½	591	608	683	2,940	3,860	4,400
87	1:2:4	592	724	714	3,445	3,900	4,210
88	1:2:4	571	678	702	2,910	3,770	4,460
89	1:2:4	769	631	711	2,755	3,430	4,200
90	1:2:4¼	568	729	695	3,380	4,010	4,760
91	1:2:4¼	652	739	732	2,935	3,890	4,520
92	1:2:4¼	577	703	666	2,590	3,320	4,180
93	1:2:4½	590	746	684	2,865	3,840	4,760
94	1:2:4½	506	769	628	2,655	3,310	4,710
95	1:2:4½	587	742	654	2,905	3,180	4,410
96	1:2:4¾	607	791	678	3,110	3,920	4,620
97	1:2:4¾	561	603	613	2,035	3,060	3,330

ROUND 1, STONE AGGREGATE, GRADING A, TYPE C FINISHER

98	1:2:3½	655	869	670	3,960	4,530	4,960
99	1:2:3½	679	807	714	3,910	4,380	4,620
100	1:2:3½	686	865	694	4,180	4,680	4,910
101	1:2:4	686	786	702	3,370	4,510	4,780
102	1:2:4	628	727	725	3,245	4,300	4,600
103	1:2:4	599	762	676	3,565	4,190	4,600
104	1:2:4¼	591	772	652	3,720	4,520	4,620
105	1:2:4¼	609	739	747	3,810	4,040	4,730
106	1:2:4¼	783	749	662	3,420	4,440	4,920
107	1:2:4½	643	715	694	3,395	4,560	5,490
108	1:2:4½	695	727	739	3,125	4,110	4,870
109	1:2:4½	588	727	670	3,055	3,640	4,070
110	1:2:4¾	549	641	712	3,330	4,160	4,630
111	1:2:4¾	553	698	690	2,940	3,600	4,670
112	1:2:4¾	532	577	672	2,030	2,740	4,220

ROUND 1, STONE AGGREGATE, GRADING B, TYPE C FINISHER

113	1:2:3½	569	613	685	3,350	3,850	4,860
114	1:2:3½	621	654	703	3,010	3,760	4,770
115	1:2:3½	514	521	644	2,790	3,620	4,550
116	1:2:4	559	719	664	3,045	4,220	4,310
117	1:2:4	608	622	602	3,370	4,180	4,800
118	1:2:4	566	635	676	2,680	3,140	4,640
119	1:2:4¼	538	613	713	2,595	2,900	4,410
120	1:2:4¼	457	686	645	3,140	3,630	4,620
121	1:2:4¼	522	672	640	3,355	4,040	4,720
122	1:2:4½	559	703	583	2,890	3,460	4,420
123	1:2:4½	484	682	670	2,330	3,070	4,200
124	1:2:4½	586	650	666	2,580	2,880	3,880
125	1:2:4¾	511	621	625	2,545	3,340	4,100
126	1:2:4¾	576	715	666	2,780	3,580	4,500
127	1:2:4¾	637	480	666	2,010	2,320	4,030

ROUND 2, GRAVEL AGGREGATE, GRADING A, TYPE C FINISHER

128	1:2:3½	469	502	540	3,415	4,050	4,520
129	1:2:3½	430	492	536	3,525	4,140	4,380
130	1:2:3½	475	466	576	3,145	3,920	4,260
131	1:2:4	443	440	543	2,905	3,760	4,350
132	1:2:4	476	530	556	3,030	3,440	4,270
133	1:2:4	412	498	440	3,260	3,920	4,760
134	1:2:4½	449	493	505	3,150	3,410	4,220
135	1:2:4½	475	516	512	3,020	3,450	4,220
136	1:2:4½	463	460	528	3,425	3,890	4,080
137	1:2:4¾	490	450	426	3,255	4,000	4,270
138	1:2:4¾	471	446	501	2,725	3,320	4,160
139	1:2:4¾	476	470	557	2,280	2,990	4,010
140	1:2:5	514	498	544	3,125	3,740	4,420
141	1:2:5	452	440	547	2,715	3,380	3,920
142	1:2:5	519	490	553	2,790	3,100	3,940
143	1:2:5¼	488	468	532	3,020	3,640	4,610
144	1:2:5¼	452	478	543	2,975	3,470	3,970
145	1:2:5¼	477	488	535	2,520	3,060	3,860

ROUND 2, GRAVEL AGGREGATE, GRADING B, TYPE C FINISHER

Section No.	Proportions	Modulus of rupture in pounds per square inch: Beams 28 days	Beams 9 months	Slabs, 9 months	Compressive strength in pounds per square inch: Cylinders 28 days	Cylinders 10 months	Cores, 15 months
146	1:2:3½	520	649	568	3,915	4,860	4,620
147	1:2:3½	516	482	579	3,585	4,550	4,370
148	1:2:3½	519	490	621	3,680	4,460	4,510
149	1:2:4	484	548	439	3,830	4,970	3,950
150	1:2:4	497	552	568	3,765	4,610	4,450
151	1:2:4	518	482	556	3,370	4,200	4,400
152	1:2:4½	450	502	541	3,065	3,670	3,840
153	1:2:4½	461	563	512	3,375	4,100	3,830
154	1:2:4½	475	526	519	3,175	4,060	4,000
155	1:2:4¾	472	526	469	3,750	4,260	4,400
156	1:2:4¾	470	535	464	3,255	4,080	3,960
157	1:2:4¾	440	482	454	2,765	3,580	3,600
158	1:2:5	426	521	540	3,220	3,980	3,840
159	1:2:5	466	497	476	2,620	3,440	3,940
160	1:2:5	440	578	561	2,715	3,760	3,720

ROUND 2, GRAVEL AGGREGATE, GRADING A, TYPE B FINISHER

161	1:2:3½	492	550	605	3,900	4,620	4,370
162	1:2:3½	526	571	641	3,610	4,380	4,480
163	1:2:3½	533	538	545	3,215	4,030	4,730
164	1:2:4	488	536	614	3,065	4,320	4,560
165	1:2:4	538	518	574	3,500	4,030	4,380
166	1:2:4	532	565	623	3,820	4,600	4,600
167	1:2:4½	509	494	609	3,150	4,000	4,940
168	1:2:4½	515	481	558	3,225	4,080	4,520
169	1:2:4½	466	495	623	2,915	3,740	4,360
170	1:2:4¾	520	540	558	3,205	3,980	4,060
171	1:2:4¾	488	516	555	3,220	4,050	3,810
172	1:2:4¾	496	536	553	2,840	3,700	4,160
173	1:2:5	465	509	567	2,020	3,000	3,720
174	1:2:5	478	[1] 391	554	2,265	2,780	3,940
175	1:2:5	456	443	524	2,860	3,680	4,420
176	1:2:5¼	439	470	578	2,515	2,980	3,800
177	1:2:5¼	422	492	514	2,130	2,840	3,980
178	1:2:5¼	476	508	540	2,325	3,440	4,130

ROUND 2, GRAVEL AGGREGATE, GRADING B, TYPE B FINISHER

179	1:2:3½	488	597	640	3,460	4,470	4,170
180	1:2:3½	490	606	552	3,800	4,880	4,480
181	1:2:3½	446	526	608	3,255	4,240	4,080
182	1:2:4	470	526	600	3,210	3,980	4,350
183	1:2:4	461	544	564	3,525	4,460	4,260
184	1:2:4	530	552	531	4,140	5,040	3,780
185	1:2:4½	475	516	574	2,780	3,640	3,840
186	1:2:4½	542	468	584	3,130	4,060	4,010
187	1:2:4½	503	523	504	3,410	4,200	4,070
188	1:2:4¾	490	546	580	2,860	3,860	3,760
189	1:2:4¾	506	494	560	3,105	4,100	4,220
190	1:2:4¾	474	508	451	3,100	3,490	4,020
191	1:2:5	432	478	545	2,250	3,240	3,770
192	1:2:5	452	486	482	2,510	3,540	3,960
193	1:2:5	399	469	510	2,525	3,710	3,680

ROUND 2, STONE AGGREGATE, GRADING A, TYPE B FINISHER

194	1:2:3½	772	706	716	3,450	4,820	4,890
195	1:2:3½	692	710	736	3,540	4,430	4,770
196	1:2:3½	639	646	748	2,975	3,830	4,740
197	1:2:4	710	700	766	2,865	4,000	4,830
198	1:2:4	662	747	702	3,170	4,340	4,686
199	1:2:4	620	696	743	2,815	3,780	4,810
200	1:2:4¼	643	602	665	2,905	3,820	4,650
201	1:2:4¼	602	634	778	2,425	3,300	4,370
202	1:2:4¼	575	628	676	3,355	4,420	4,760
203	1:2:4½	632	568	741	2,845	3,660	4,620
204	1:2:4½	626	590	685	2,910	4,000	4,730
205	1:2:4½	620	714	684	3,185	3,980	4,860
206	1:2:4¾	618	718	683	2,340	3,260	4,140
207	1:2:4¾	633	694	731	2,680	3,560	4,280
208	1:2:4¾	593	711	702	2,760	3,660	4,380

ROUND 2, STONE AGGREGATE, GRADING B, TYPE B FINISHER

209	1:2:3½	604	597	727	2,890	4,110	4,360
210	1:2:3½	602	676	714	3,000	4,080	4,580
211	1:2:3½	703	746	670	3,300	4,290	4,740
212	1:2:4	620	749	686	2,750	3,260	4,390
213	1:2:4	593	688	716	2,710	3,460	4,120
214	1:2:4	616	760	716	2,950	3,780	5,000
215	1:2:4¼	577	704	736	2,070	2,840	4,080
216	1:2:4¼	565	737	708	2,740	3,600	4,350
217	1:2:4¼	590	714	638	3,000	4,150	4,260
218	1:2:4½	576	621	696	1,830	2,830	3,660
219	1:2:4½	566	592	716	2,020	2,560	4,200
220	1:2:4½	609	772	703	2,700	3,570	4,200
221	1:2:4¾	565	700	734	1,590	2,040	3,670
222	1:2:4¾	622	756	701	2,410	3,000	4,400
223	1:2:4¾	632	750	651	2,900	3,400	4,700

[1] One beam only.

TABLE 6.—*Results of strength tests*—Continued

ROUND 2, STONE AGGREGATE, GRADING A, TYPE C FINISHER

Section No.	Proportions	Modulus of rupture in pounds per square inch: Beams 28 days	Beams 9 months	Slabs, 9 months	Compressive strength in pounds per square inch: Cylinders 28 days	Cylinders 10 months	Cores, 15 months
224	1:2:3½	603	680	691	2,750	3,500	4,420
225	1:2:3½	567	735	736	2,890	3,940	4,460
226	1:2:3½	603	686	730	2,930	3,820	4,660
227	1:2:4	646	706	752	2,700	3,320	4,180
228	1:2:4	598	682	691	2,870	3,640	4,420
229	1:2:4	593	684	700	2,920	4,380	4,450
230	1:2:4¼	584	812	662	3,220	4,410	4,800
231	1:2:4¼	588	790	723	2,900	4,060	4,410
232	1:2:4¼	543	772	737	2,480	3,760	4,500
233	1:2:4½	614	651	687	2,980	4,140	4,580
234	1:2:4½	586	812	701	2,690	3,820	4,300
235	1:2:4½	504	728	650	2,430	3,360	4,200
236	1:2:4¾	556	656	700	2,880	4,020	4,290
237	1:2:4¾	590	716	665	2,510	3,720	4,320
238	1:2:4¾	557	626	772	2,140	2,930	3,940

ROUND 2, STONE AGGREGATE, GRADING B, TYPE C FINISHER

239	1:2:3½	660	802	726	3,170	4,260	4,690
240	1:2:3½	577	684	761	2,850	4,020	4,320
241	1:2:3½	662	631	690	2,540	3,520	4,840
242	1:2:4	636	772	739	3,060	3,860	5,150
243	1:2:4	608	678	702	2,700	3,770	4,880
244	1:2:4	623	686	708	2,430	3,260	4,720
245	1:2:4¼	629	698	691	2,720	3,840	5,180
246	1:2:4¼	578	704	726	2,190	3,570	4,760
247	1:2:4¼	665	762	605	3,290	4,440	5,000
248	1:2:4½	664	696	726	2,350	3,080	4,900
249	1:2:4½	656	711	675	2,730	3,200	5,080
250	1:2:4½	553	596	724	2,090	2,770	4,690
251	1:2:4¾	610	698	611	2,770	3,120	4,930
252	1:2:4¾	554	675	700	2,370	3,120	4,760
253	1:2:4¾	565	734	701	2,360	2,920	4,760

ROUND 1, SLAG AGGREGATE, GRADING A, TYPE C FINISHER

254	1:2:3½	623	618	650	3,160	4,290	4,860
255	1:2:3½	598	620	679	3,300	4,040	4,940
256	1:2:3½	604	584	603	3,740	4,500	4,890
257	1:2:4	598	586	642	3,480	4,660	4,880
258	1:2:4	579	630	592	3,080	4,220	4,190
259	1:2:4	603	623	668	2,620	4,150	4,440
260	1:2:4¼	553	616	635	2,830	3,860	4,660
261	1:2:4¼	551	624	636	2,530	3,760	4,760
262	1:2:4¼	618	646	650	3,130	4,460	5,110
263	1:2:4½	570	593	592	2,770	3,780	4,660
264	1:2:4½	516	580	653	2,600	3,520	4,700
265	1:2:4½	547	620	576	2,950	4,300	4,710

TABLE 7.—*Water-cement ratio, cement factor, and strength tests* [1]

GRAVEL, GRADING A

Proportions	Water-cement ratio by volume	Theoretical cement factor in sacks per cubic yard	Modulus of rupture in pounds per square inch: Beams 28 days	Beams 9 months	Slabs, 9 months	Compressive strength in pounds per square inch: Cylinders 28 days	Cylinders 10 months	Cores, 15 months
1:2:3½	0.86	5.53	492	567	565	3,350	4,240	4,730
1:2:4	.91	5.15	472	554	547	3,180	4,000	4,590
1:2:4½	.92	4.84	473	499	541	3,040	3,900	4,500
1:2:4¾	.96	4.67	467	490	530	2,880	3,600	4,230
1:2:5	.98	4.52	458	477	540	2,680	3,300	4,180
1:2:5¼	.97	4.41	435	483	516	2,630	3,380	4,140

GRAVEL, GRADING B

1:2:3½	0.89	5.54	506	582	578	3,620	4,520	4,570
1:2:4	.92	5.17	494	557	553	3,520	4,420	4,420
1:2:4½	.99	4.81	486	541	532	3,260	4,040	4,200
1:2:4¾	1.01	4.67	468	525	509	3,140	3,940	4,150
1:2:5	1.05	4.51	439	536	494	2,800	3,690	4,080
1:2:5¼ [2]	1.02	4.42	466	558	475	3,140	4,000	3,980

STONE, GRADING A

1:2:3½	0.87	5.73	654	754	715	3,480	4,330	4,850
1:2:4	.91	5.35	648	730	706	3,240	4,180	4,700
1:2:4¼	.92	5.18	615	744	700	3,160	4,030	4,680
1:2:4½	.94	5.02	614	718	694	3,050	3,960	4,700
1:2:4¾	.97	4.86	556	694	695	2,660	3,540	4,370

STONE, GRADING B

1:2:3½	0.87	5.78	600	658	700	3,060	4,000	4,620
1:2:4	.91	5.39	614	695	694	2,900	3,680	4,570
1:2:4¼	.92	5.22	576	705	683	2,840	3,690	4,570
1:2:4½	.95	5.04	578	690	677	2,500	3,150	4,430
1:2:4¾ [3]	.97	4.90	584	684	668	2,450	3,070	4,350

SLAG, GRADING A

1:2:3½	0.87	5.90	608	607	644	3,400	4,280	4,900
1:2:4	.93	5.50	593	613	634	3,060	4,340	4,500
1:2:4¼	.95	5.33	574	629	640	2,830	4,030	4,840
1:2:4½	.99	5.14	544	598	607	2,770	3,870	4,690

[1] Except where otherwise indicated, each value is the average for 12 sections in the case of gravel and stone and 3 sections in the case of slag.
[2] 2 sections only.
[3] 11 sections only.

RELATION BETWEEN STRENGTH OF CONCRETE AND MIX

The average results of strength tests on the concrete for each grading of aggregate for each mix are shown in Table 7. The average results of all tests for each aggregate are plotted in graphic form in Figures 10, 11, and 12. Each plotted value in these figures represents the average of all tests for the particular mix and type of aggregate indicated. In the case of gravel and stone, each point therefore represents the average of tests on 24 sections, except the 1:2:5¼ mix, gravel, which represents 14 sections, and the 1:2:4¾ mix, stone, which represents 23 sections. In the case of slag, each point represents the average of tests on only three sections. Twenty-eight day and nine-month tests on control specimens for each section are the average of tests on two beams and two cylinders. The tests on the slabs are the average of the four beams taken from the section and the tests on the cores are the averages of tests of two cores drilled from the section. Each point, therefore, with the exceptions noted above, becomes the average of 48 individual tests in the case of the beams, cores, and cylinders and the average of 96 individual tests in the case of the pavement slabs.

Let us first examine Figure 10, on which the results for the gravel concrete are plotted. A fairly consistent decrease in both flexural and compression strength is noted for increases in the percentages of coarse aggregate in the mix. It will be observed, however, that, whereas the flexure specimens show about the same percentage of decrease for the slabs at nine months as for the beams at 28 days and 9 months, the cores tested in compression at 15 months show a somewhat smaller rate of decrease than the cylinders at either 28 days or 10 months. It will be observed also that the percentage of increase in strength from 28 days to 9 months in the case of the beams and from 28 days to 10 months in the case of the cylinders is approximately the same for all mixes, although the average percentage of increase is greater in the case of the compression tests. A very close agreement is shown between the results of beam and slab tests at nine months for all mixes. The data indicate that in general the results of tests on beam control specimens may be considered to represent very closely the strengths of the slabs. However, these are average relations and it will be shown later during the discussion of the individual test results that such close agreement does not apply under all conditions.

Unfortunately it was impossible to test the cores at the same age as the 10-months cylinders, so that an accurate comparison in compression can not be made. The observed increase in cylinder strengths from 28 days to 10 months would indicate, at least in the case of the richer mixtures, a probable further increase to approximately the strengths obtained from the cores at 15 months. It is difficult to predict just what the rela-

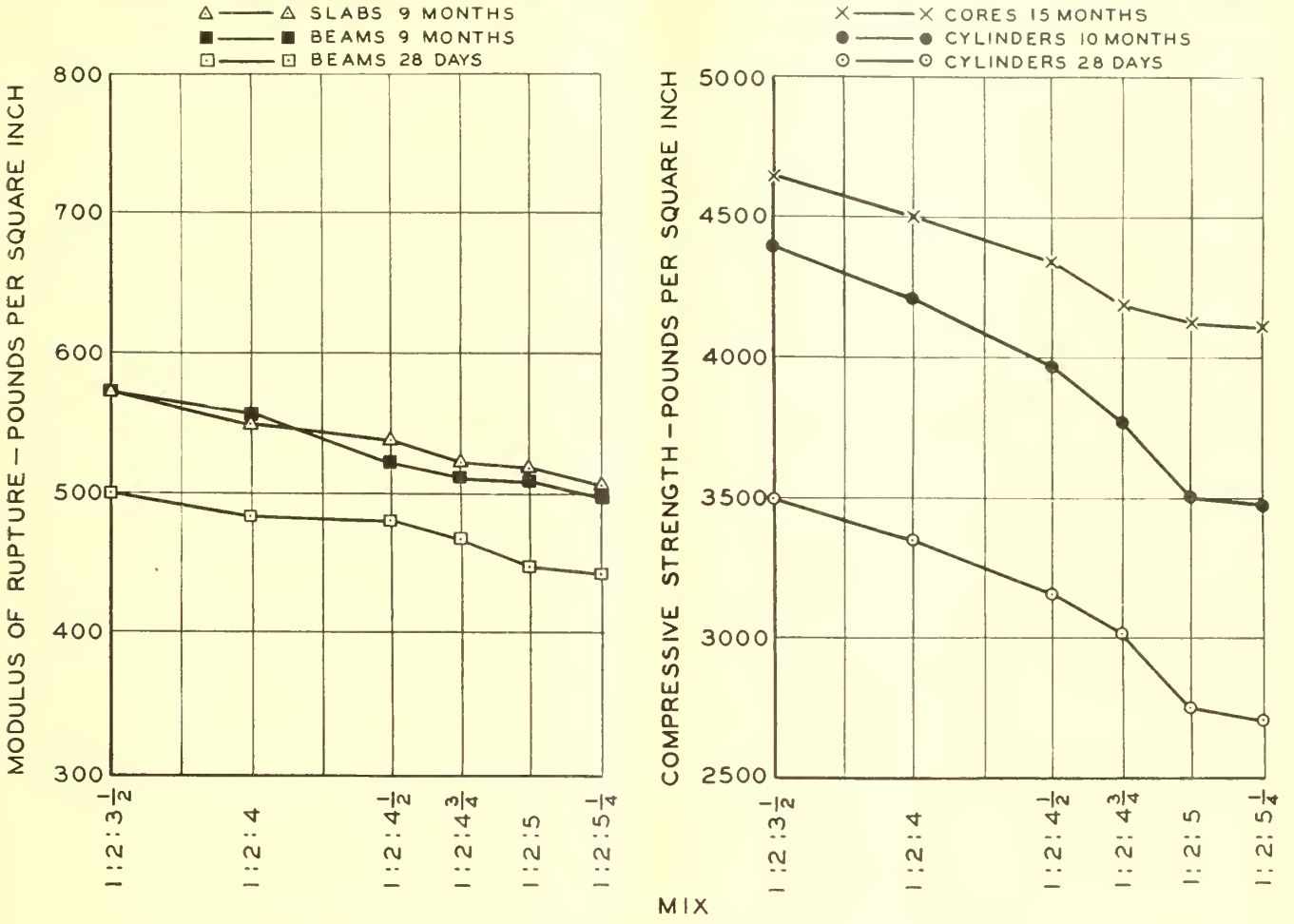


FIGURE 10.—RELATION BETWEEN STRENGTH OF GRAVEL CONCRETE AND MIX

tion would have been for the mixes containing larger quantities of coarse aggregate, because of the fact that the cylinder strengths decreased with increase of coarse aggregate at a somewhat greater rate than the core strengths. In general, the data from the gravel concrete group seem to indicate that the results of tests on molded cylinders are a fair indication of the crushing strength of the concrete in the pavement.

The corresponding values for the crushed stone concrete are shown in Figure 11. The same general trends are noted as in the case of the gravel concrete. The average reduction in flexural strength for a corresponding increase in coarse aggregate content does not, however, appear to be as great. For instance, in the case of the crushed-stone slabs, an increase in coarse aggregate from 3½ to 4¾ parts resulted in a decrease in modulus of rupture of only 26 pounds per square inch, or approximately 4 per cent. (See also Table 7.) The corresponding reduction for the gravel concrete was 52 pounds per square inch, or about 9 per cent. A similar increase in the amount of coarse aggregate reduced the crushing strength of the stone concrete cores 380 pounds per square inch, or 8 per cent, and the gravel concrete 460 pounds per square inch, or 10 per cent. There is not the same agreement between results of 9-month beam and slab strengths for the stone concrete as for the gravel concrete. This is particularly true of the 1:2:4¼ mix. In the case of stone concrete the relation between flexural strength and mix is more consistent for the slabs than for the beams, possibly because of the difficulty of molding uniform beam specimens of comparatively small section (7 inches by 7 inches) when an angular aggregate is used.

Comparing now the results of compression tests on the stone concrete, as shown in Figure 11, with the corresponding values in Figure 10, one will observe that the difference between cylinder strengths at 10 months and core strengths at 15 months is considerably greater than for the gravel concrete, although, as in the case of the gravel concrete, the rate of decrease with increased percentages of coarse aggregate is greater for the cylinders than for the cores. Since the average curing conditions and other factors which might affect the rate of increase in strength were similar, it seems reasonable to assume, in the case of the stone concrete, that the cylinders would not have increased in strength between 10 and 15 months at any greater rate than the gravel concrete. This would indicate the possibility that in the case of the crushed stone concrete, the core strengths at a given age might have been somewhat higher than the control cylinder strengths. The difference, if any, would have been relatively small.

Relations between the strength of the slag concrete and mix are shown in Figure 12. As has been indicated, each value in this case represents the average of tests on three slabs only, the only variable for each proportion being the consistency. Type C finisher only was used with A grading for the slag. In spite of the fact that the results are not so consistent as for the other aggregates, the same general trends may be observed. The

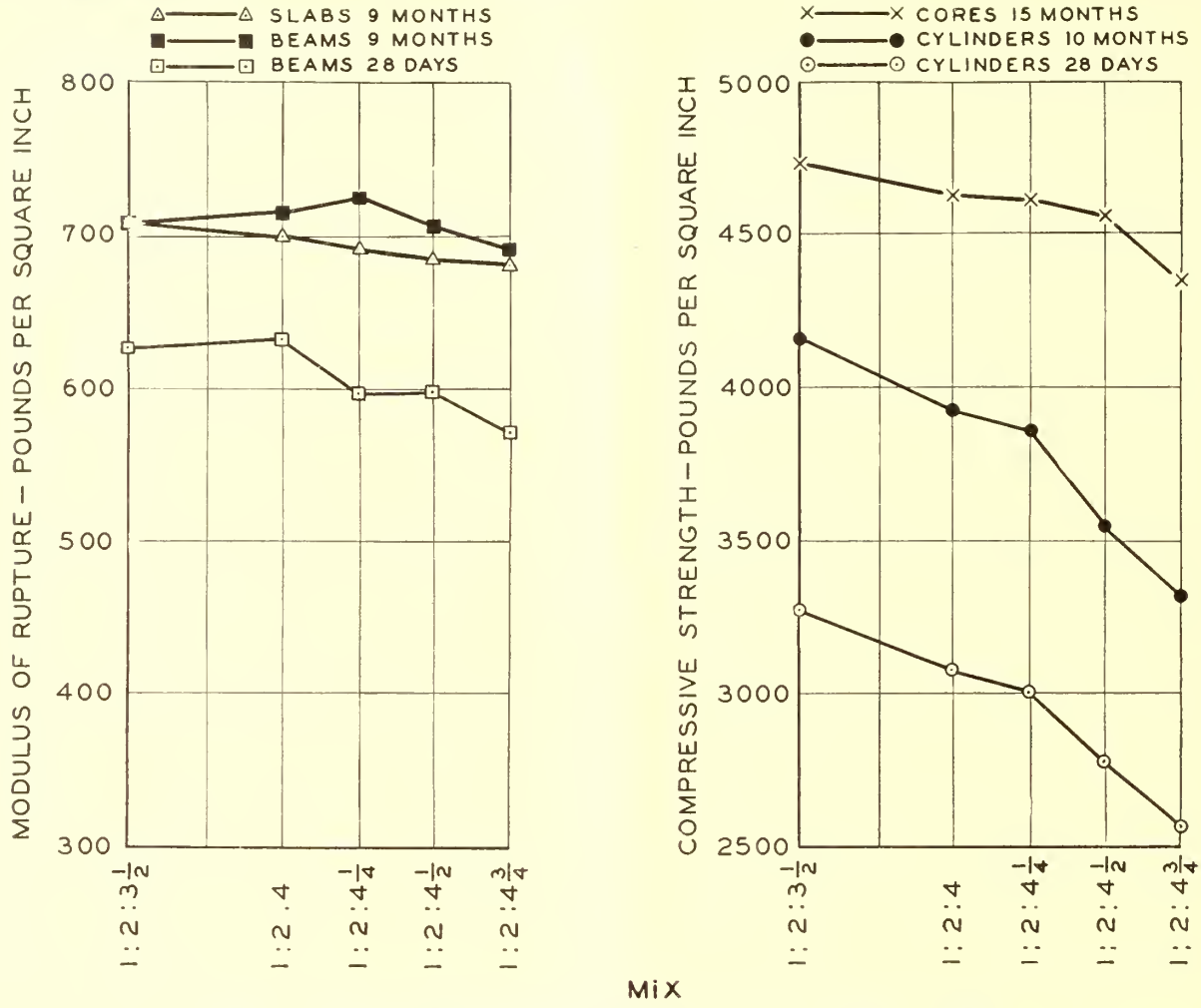


FIGURE 11.—RELATION BETWEEN STRENGTH OF STONE CONCRETE AND MIX

most puzzling deviations are the beam tests at 9 months for the 1:2:3½ and 1:2:4 mixes. There appears to be no explanation for these low values, nor for the erratic results in compression for the 1:2:4 mix at 10 months and 15 months. It seems reasonable to assume that these erratic results are purely accidental and that much more consistent values would have been obtained had a larger number of specimens been tested.

RELATION BETWEEN STRENGTH OF CONCRETE AND CEMENT FACTOR

The average results for flexural and compressive strength which have just been discussed from the standpoint of mix are plotted in Figures 13, 14, and 15 against the cement content in bags per cubic yard of concrete resulting from the arbitrary proportions which were used. It seemed desirable to study the data from this standpoint because of the difference in cement factor for a given nominal mix caused by difference in the voids in the coarse aggregate. Reference to Table 2 will show that, for the same grading, the gravel contained 5 per cent less voids than the crushed stone and 9 per cent less voids than the crushed slag. These differences are reflected in a variation in yield of concrete to the extent of approximately 0.2 bag per cubic yard in the case of gravel compared with crushed stone and about 0.35 bag per cubic yard in the case of gravel compared with slag. (See also Table 7.) This difference is, of course, due to the fact that the base proportions were determined by bulk volume. For identical proportions by absolute volume the yields would have been the same regardless of the difference in void content of the different aggregates.

Certain interesting relationships may be pointed out. For instance, from Figures 13 and 14 it is seen that for the flexure specimens and for the cores the decrease in strength with reductions in cement content progresses at a fairly uniform rate, whereas in the case of the cylinders, the rate of reduction in the strength of the concrete increases as the cement content is reduced. This applies to both the gravel and the stone concrete. For a variation in coarse aggregate from 3½ to 4¾ parts, which is the extreme range for the stone concrete, the average decrease in flexural strength of slab (the average including both stone and gravel concrete) is about 6 per cent (see Table 7). The corresponding average decrease in cement content is approximately 15 per cent. The same increase in coarse aggregate reduced the average crushing strength of the cores about 9 per cent with the same decrease in cement factor. Attention is called to the fact that the reduction in strength with increasing quantities of coarse aggregate in the mix is, according to the water-cement ratio theory, caused by the dilution of the cement paste, which in turn is brought about by increasing the water content in order to maintain workability. This phase of the matter is discussed in the next section.

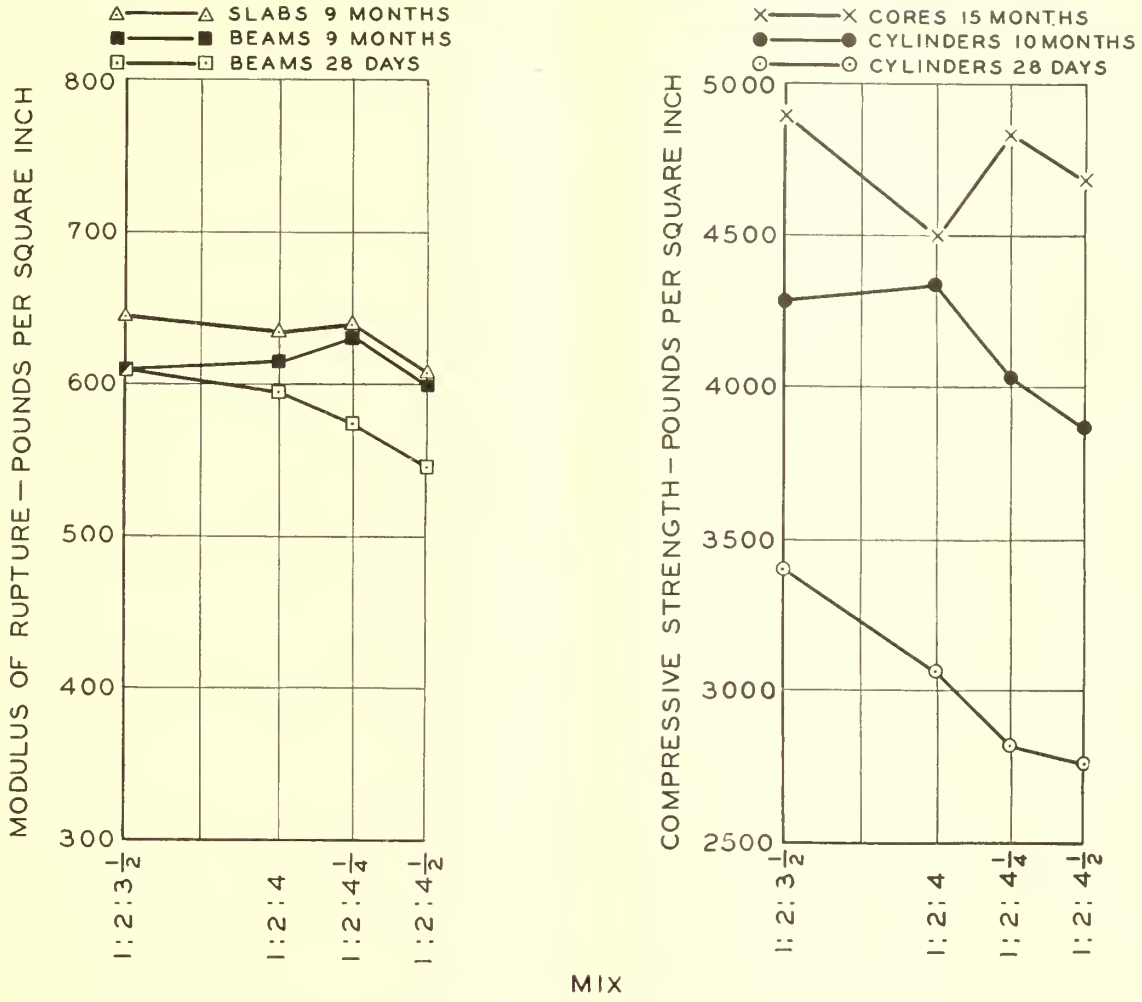


FIGURE 12.—RELATION BETWEEN STRENGTH OF SLAG CONCRETE AND MIX

RELATION BETWEEN STRENGTH AND WATER-CEMENT RATIO

The effect on strength of change in the water-cement ratio brought about by increasing the quantity of coarse aggregate is shown in Figures 16 and 17. The values for water-cement ratio indicated in these charts are approximate net water ratios after correctng for absorption of water by the sand but without making any correction for absorption by the coarse aggregate. The coarse aggregates were assumed to be in a saturated surface-dry condition when used and hence no correction should be made in calculating the net water ratio. On the other hand the total amount of water used in the concrete as reported in Table 7 was determined by adding the total water carried by the sand to the quantity introduced at the mixer. This would of course require a correction to take care of absorption in the sand. Based on 0.7 per cent absorption this correction amounts approximately to 0.02 so that the net water-cement ratios shown on the charts are 0.02 lower than the average values calculated from Table 7.

It was found impossible to determine accurately the absorption in the slag. Niether does it appear possible to assume that the slag was in a saturated surface dry condition when used. For these reasons values for net water ratio for the slag concrete were not calculated and no chart is shown for this material.

Examination of Figures 16 and 17 reveals the conventional relation between water-cement ratio and strength. It will be observed also that, for a given water-cement ratio, the crushing strengths of the stone concrete cores are approximately the same as those of the gravel concrete cores, whereas, in the case of the control cylinders, the gravel concretes show considerably higher crushing strengths than the stone concretes. The difference appears also to become greater as the water-cement ratio increases.

In flexure the crushed stone concrete, for equivalent water ratios, is considerably higher in both beam and slab strength, probably because of certain aggregate characteristics inherent in these particular materials.

EFFECT OF TYPE OF FINISHING MACHINE

The average values for modulus of rupture of the slabs and crushing strength of the cores have been plotted by types of finishing machine in Figures 18 and 19. The data were taken from Tables 8 and 9. In these charts each plotted value represents the average of tests on six test sections, in the case of types A and B finisher, and twelve test sections in the case of type C finisher, except for the 1:2:5¼ mix, gravel concrete, and the 1:2:4¾ mix, stone concrete.

According to these curves, the type B finisher gives higher results in flexure than either type A or type C. In compression on the other hand, type A is high and type B low. These comparisons apply to both types of aggregate although the high values for both flexure and compression are more pronounced in the case of the gravel concrete than for the stone. It will be re-

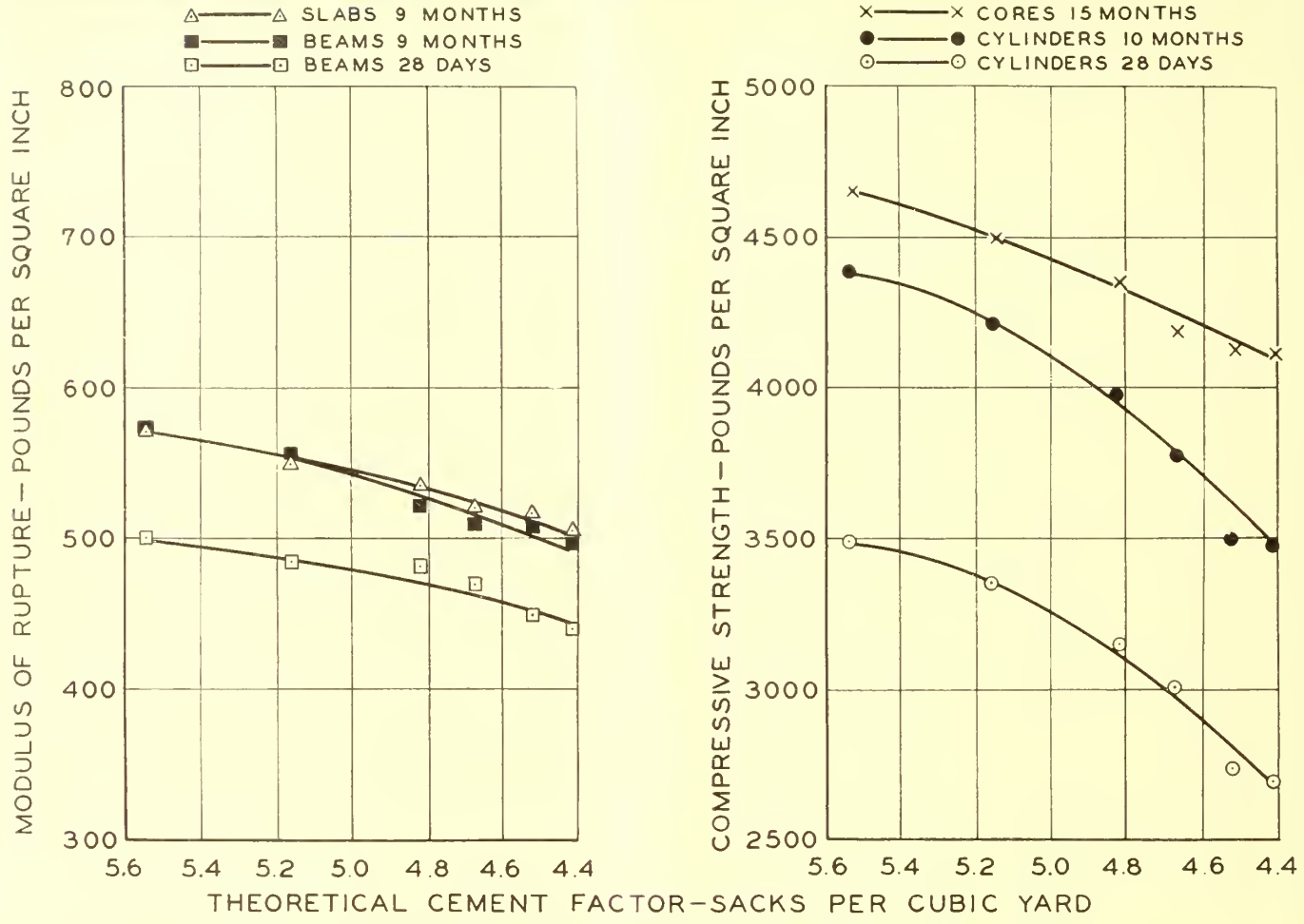


FIGURE 13.—RELATION BETWEEN STRENGTH OF GRAVEL CONCRETE AND CEMENT FACTOR

called that the flexure tests were made on the slabs with the load applied on top and with the bottom in tension. It seems rather difficult to explain just why type B finisher should have given values for modulus of rupture higher than either of the other types. This, it will be recalled, was a single-screed machine and there seems at first sight no reason why it should have compacted the concrete any more thoroughly than the double screed. However, the screed on the type B machine was tilted slightly by raising the front of the screed about one-half inch, whereas the screeds on the type C machine were flat. The tilted screed might possibly have a tendency to compact the concrete slightly more than a screed of the type C design.

In order to determine whether the relatively high values of modulus of rupture for type B finisher might have been due to some other factor, as for instance more favorable curing conditions, the average results of the slab tests for each mix, grading, and type of finisher have been compared directly with the average results

TABLE 8.—*Effect of type of finishing machine on strength of gravel concrete* [1]

MODULUS OF RUPTURE OF SLABS AT 9 MONTHS, IN POUNDS PER SQUARE INCH

Grading	Proportions	Finisher used			
		Type A, round 1	Type B, round 2	Type C	
				Round 1	Round 2
A	1:2:3½	540	597	571	551
	1:2:4	535	604	535	513
	1:2:4½	549	597	506	515
	1:2:4¾	530	557	538	495
	1:2:5	549	548	513	548
	1:2:5¼	496	544	487	537
B	1:2:3½	593	600	530	589
	1:2:4	554	565	573	521
	1:2:4½	542	554	506	524
	1:2:4¾	504	530	540	[2] 462
	1:2:5	465	512	475	526
	1:2:5¼			[3] 475	

COMPRESSIVE STRENGTH OF CORES AT 15 MONTHS, IN POUNDS PER SQUARE INCH

Grading	Proportions	Finisher used			
		Type A, round 1	Type B, round 2	Type C	
				Round 1	Round 2
A	1:2:3½	5,010	4,530	5,000	4,390
	1:2:4	4,580	4,510	4,820	4,460
	1:2:4½	4,800	4,610	4,440	4,170
	1:2:4¾	4,370	4,010	4,380	4,150
	1:2:5	4,240	4,030	4,370	4,090
	1:2:5¼	4,080	3,970	4,370	4,150
B	1:2:3½	5,090	4,240	4,450	4,500
	1:2:4	4,850	4,130	4,420	4,270
	1:2:4½	4,520	3,970	4,400	3,890
	1:2:4¾	4,480	4,000	4,120	3,990
	1:2:5	4,690	3,800	4,000	3,830
	1:2:5¼			[4] 3,980	

[1] Except where otherwise indicated, each value is the average for 12 specimens in the case of modulus of rupture and 6 specimens in the case of compressive strength.
[2] Average of 11 specimens.
[3] Average of 8 specimens.
[4] Average of 4 specimens.

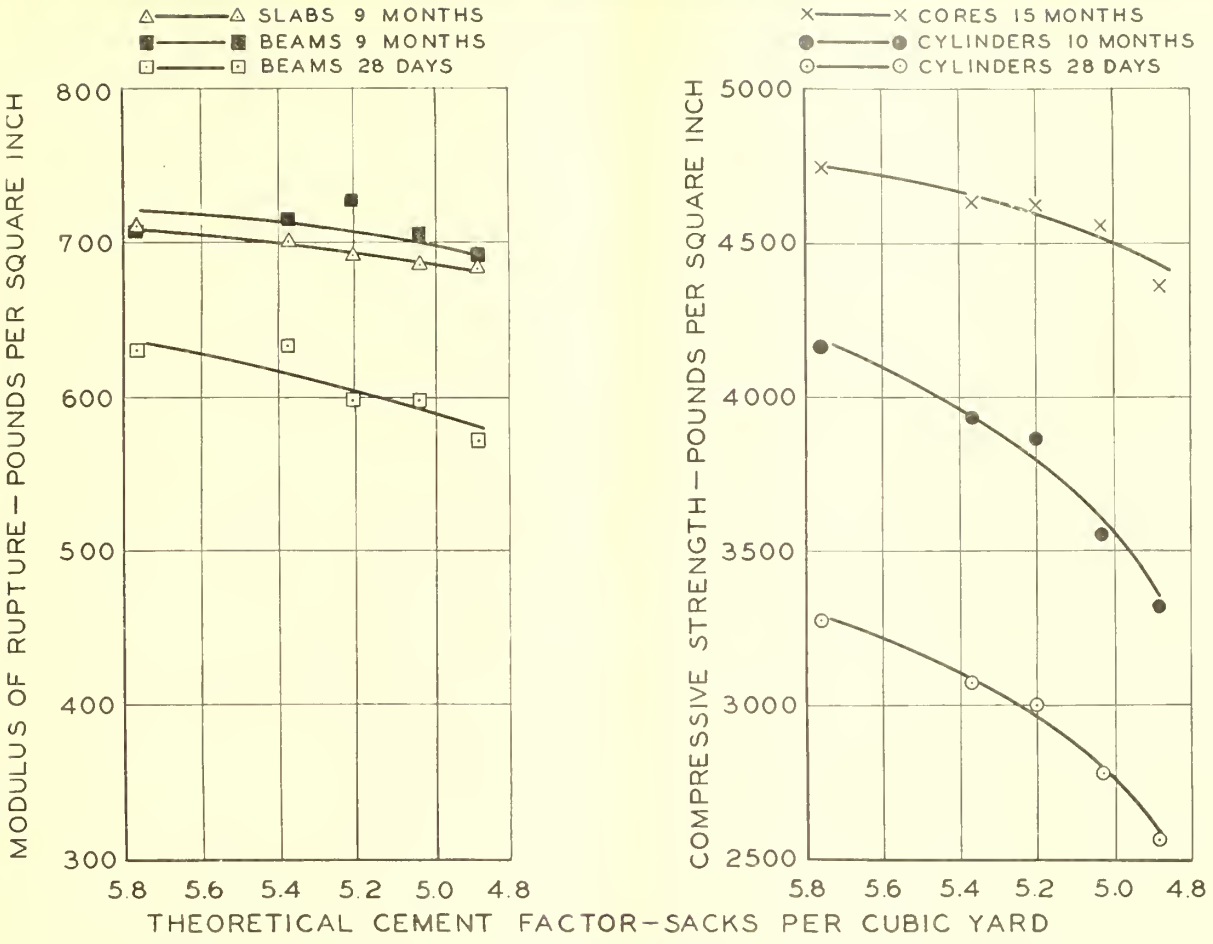


FIGURE 14.—RELATION BETWEEN STRENGTH OF STONE CONCRETE AND CEMENT FACTOR

TABLE 9.—*Effect of type of finishing machine on strength of stone concrete* [1]

MODULUS OF RUPTURE OF SLABS AT 9 MONTHS, IN POUNDS PER SQUARE INCH

Grading	Proportions	Finisher used			
		Type A, round 1	Type B, round 2	Type C	
				Round 1	Round 2
A	1:2:3½	715	733	693	719
	1:2:4	672	737	701	714
	1:2:4¼	698	706	687	707
	1:2:4½	694	703	701	679
	1:2:4¾	672	705	691	712
B	1:2:3½	693	704	[2] 677	726
	1:2:4	709	706	647	716
	1:2:4¼	698	694	666	674
	1:2:4½	655	705	640	708
	1:2:4¾	[3] 646	695	652	671

COMPRESSIVE STRENGTH OF CORES AT 15 MONTHS, IN POUNDS PER SQUARE INCH

Grading	Proportions	Finisher used			
		Type A, round 1	Type B, round 2	Type C	
				Round 1	Round 2
A	1:2:3½	5,270	4,800	4,830	4,510
	1:2:4	5,020	4,770	4,660	4,350
	1:2:4¼	4,820	4,590	4,760	4,570
	1:2:4½	4,890	4,740	4,810	4,360
	1:2:4¾	4,510	4,270	4,510	4,180
B	1:2:3½	4,550	4,560	4,730	4,620
	1:2:4	4,290	4,500	4,580	4,920
	1:2:4¼	4,490	4,230	4,580	4,980
	1:2:4½	4,630	4,020	4,170	4,890
	1:2:4¾	[4] 3,980	4,260	4,210	4,820

[1] Except where otherwise indicated, each value is the average for 12 specimens in the case of modulus of rupture and 6 specimens in the case of compressive strength.
[2] Average of 11 specimens.
[3] Average of 8 specimens.
[4] Average of 4 specimens.

for the corresponding control beams. The "strength ratios," obtained by dividing the average slab strength by the average beam strength are used to express the relative efficiency of the different finishers. These "strength ratios" are given in Table 10 for each type of machine, type of aggregate, and grading of aggregate.

It will be observed that in the case of the gravel concrete and also in the case of the stone concrete, grading A, type B finisher is still high. However, there appears to be no trend in the case of the stone concrete, grading B. Another point of interest is that in the case of the gravel concrete, grading A, there is a tendency for the slabs to show higher strength in proportion to the beams with increasing amounts of coarse aggregate. This applies to all types of finisher. On the other hand, for the gravel concrete, grading B, the reverse seems to be true except for finisher B. The tendency for the gravel concrete, grading B, to honeycomb, particularly in the leaner mixes, with consequent falling off in slab strength, probably accounts for this. The effect of honeycombing on strength is discussed later.

In the case of the stone concrete there appears to be very little trend as regards the effect of increasing percentages of coarse aggregate except that for grading B the 1:2:3½ mix seems to give relatively high strength ratios for all three types of finisher.

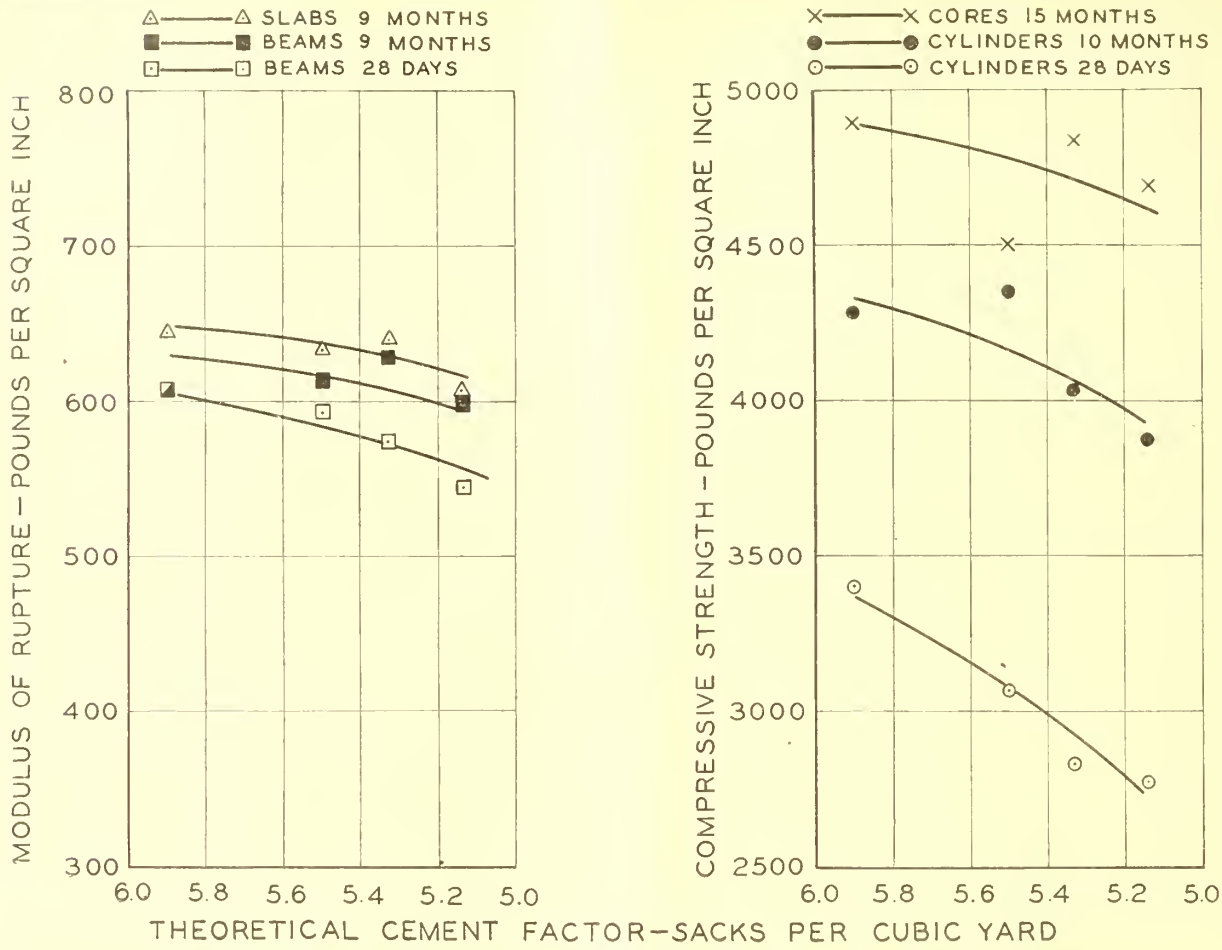


FIGURE 15.—RELATION BETWEEN STRENGTH OF SLAG CONCRETE AND CEMENT FACTOR

TABLE 10.—*Ratio of flexural strengths of slabs to strengths of corresponding control beams at 9 months* [1]

Type of finisher	Coarse aggregate	Proportions							Average
		1:2:3½	1:2:4	1:2:4¼	1:2:4½	1:2:4¾	1:2:5	1:2:5¼	
A	Gravel-A	0.99	1.06	.....	1.07	1.15	1.19	1.07	1.09
B	do	1.08	1.12	.....	1.22	1.05	1.19	1.11	1.13
C	do	.99	.92	.....	1.03	1.07	1.08	1.05	1.02
Average	do	1.02	1.03	.....	1.11	1.09	1.15	1.08	1.08
A	Gravel-B	.94	.89	.....	.93	.95	.78	.....	.90
B	do	1.04	1.04	.....	1.10	1.03	1.07	.....	1.06
C	do	1.00	1.03	.....	.96	.95	.94	.....	.98
Average	do	.99	.99	.....	1.00	.98	.93	.....	.98
A	Stone-A	.91	.86	.86	.87	.88	.....	.....	.88
B	do	1.07	1.03	1.14	1.13	1.00	.....	.....	1.07
C	do	.93	.98	.90	.95	1.08	.....	.....	.97
Average	do	.97	.96	.97	.98	.99	.....	.....	.97
A	Stone-B	1.06	1.05	.96	.87	.93	.....	.....	.97
B	do	1.05	.96	.97	1.06	.95	.....	.....	1.00
C	do	1.09	1.00	.98	1.00	1.02	.....	.....	1.02
Average	do	1.07	1.00	.97	.98	.97	.....	.....	1.00

[1] Values obtained by averaging ratios given by individual sections.

In general it may be said that, aside from a slight advantage in favor of the type B finisher, the analysis of the data indicates that the three types of finishing machine give essentially the same results and also that beam strength is a fairly good indication of the actual flexural strength of the concrete in the structure. In this connection, however, attention should be called to the discussion on the effect of honeycombing which is given in a subsequent part of the report. In this discussion the fact is brought out that this relation does not apply in the case of very dry mixes which honeycomb on placing.

In connection with the discussion on finishing, Table 12 gives the number of passes of the finisher and, in the case of type A, the number of passes of the tamper for each section, together with notes on workability and finish made during construction. These data may be used in connection with a study of the detailed test results plotted in Figures 33 to 37.

EFFECT OF GRADING OF COARSE AGGREGATE

The average effect of grading of the coarse aggregate upon the slab and core strengths of the concrete is shown in Figures 20 and 21. The data were taken from Table 7. The concrete containing the well-graded coarse aggregate shows somewhat higher strength values than that containing the more poorly graded aggregate, although the differences are not very great. For mixes containing more than 4½ parts coarse aggregate there appears to be a tendency for the concrete containing the small size gravel (grading B) to decrease in flexural strength at a greater rate with increasing amounts of aggregate than the large-size material. As has been noted, this is no doubt due to the tendency of the poorly graded concrete to honeycomb in the leaner mixes. Attention should also be called to the fact that a wider variation in strength results from changes in consistency in a given mix than from the average variations due to changes in grading. This may be verified by analyzing the data for the individual sec-

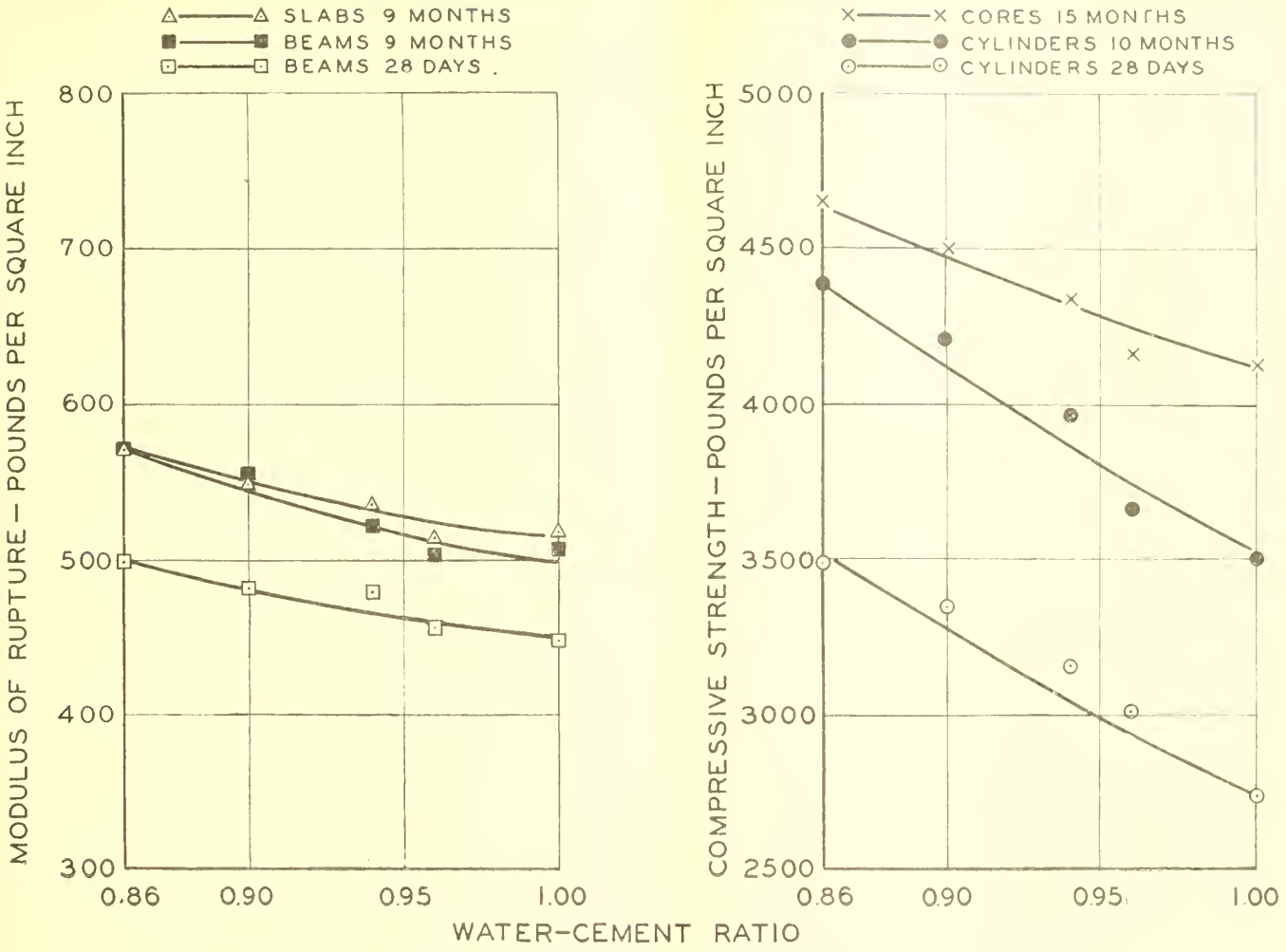


FIGURE 16.—RELATION BETWEEN STRENGTH OF GRAVEL CONCRETE AND WATER-CEMENT RATIO

tions as shown in Table 6 and in Figures 33 to 37, inclusive, as well as in Figures 25, 26, and 27. This is particularly true for the mixes containing from 3½ to 4½ parts coarse aggregate. For the mixes high in coarse aggregate the tendency of grading B aggregate to produce honeycomb results in a considerable difference due to this cause.

FACTORS INFLUENCING HONEYCOMB IN SLABS STUDIED

In order to analyze the manner in which honeycombing is affected by the several variables which have been introduced into this study, as well as to bring out the effect of honeycombing in slabs upon flexural strength, a series of charts have been prepared which will be discussed in the following section of this report.

The percentage of honeycomb in each test slab was determined by averaging two values, the percentage of the total area of the bottom of the slab showing honeycomb and the percentage of the total area of the cross section at the break which showed honeycomb. It was felt that this method gave a reasonably satisfactory measure of the homogenity of the concrete in the test slab.

The relation between average percentages of honeycomb in each mix for each type and gradation of coarse aggregate is shown in Figure 22; and the corresponding average relation between honeycomb and consistency as indicated by slump tests, is shown in Figure 23. A similar relation between honeycomb and consistency but with the values classified by types of finisher is shown in Figure 24. In these figures each point represents the average of the percentages of honeycomb in all the sections having the particular combination of variables involved. The data are taken from Table 13, which shows the percentage of honeycomb for each section and Table 5, which gives the slump and flow test values for each section. In the case of Figures 22 and 23, the plotted points do not always represent the average of tests on the same number of slabs because of the fact that the same slumps were not always obtained on each of the three sections in each group. In drawing in the average lines each point was weighted in accordance with the number of tests which it represented.

These figures are of interest in showing (1) that the average amount of honeycomb increases slightly with increases in the percentage of coarse aggregate in the mix (fig. 22); (2) that where the small-size gravel aggregate containing an excess of fine material was used the average amount of honeycomb was considerably increased (fig. 22); (3) that the average amount of honeycomb was increased by decreasing the slump (fig. 23); and (4) that honeycombing was somewhat decreased by the use of type A finishing machine (fig. 24).

One of the most interesting features in connection with the occurrence of honeycomb in the concrete is the effect of B grading in the gravel mixes. Reference

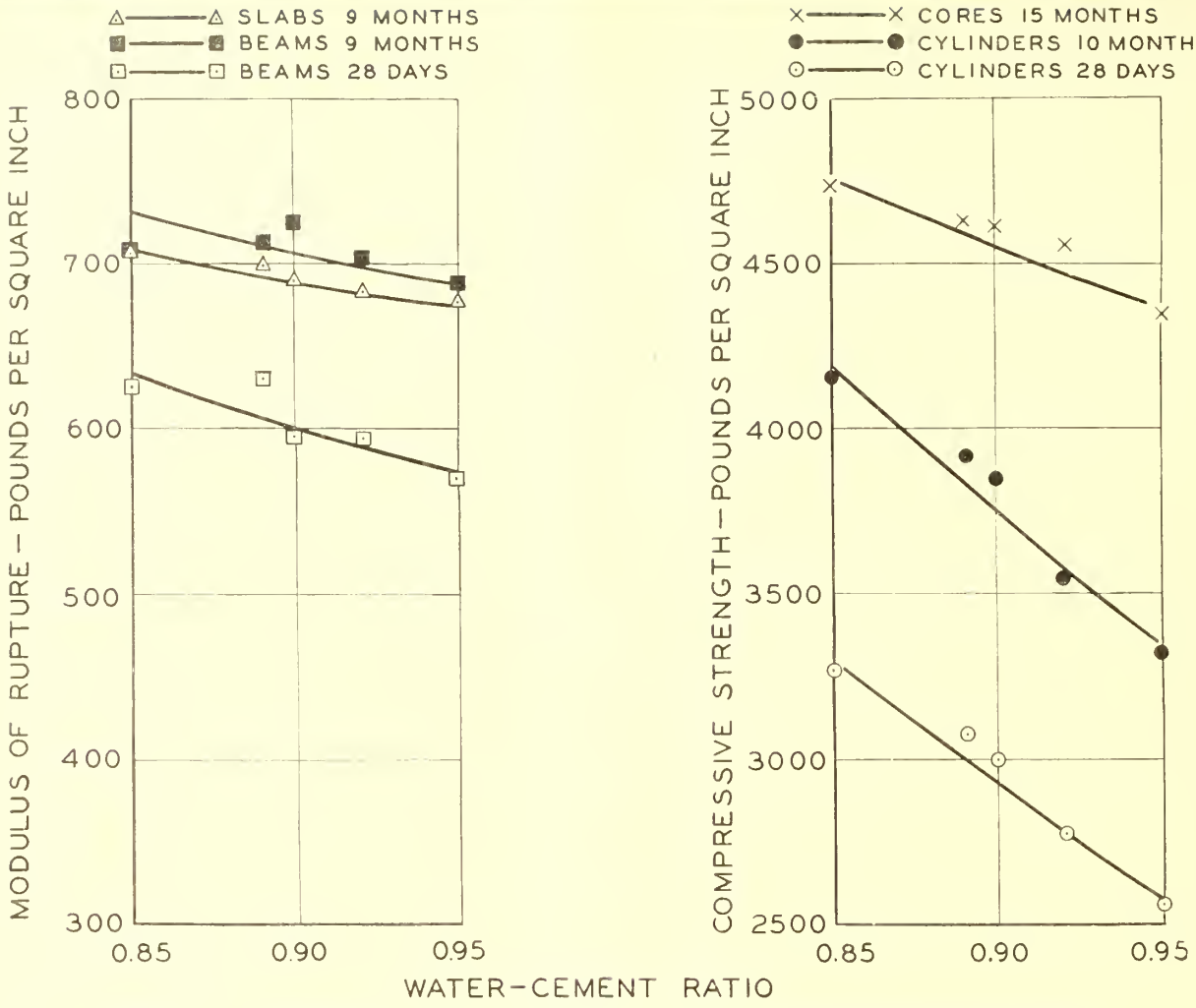


FIGURE 17.—RELATION BETWEEN STRENGTH OF STONE CONCRETE AND WATER-CEMENT RATIO

to Figure 1 will show that this material was graded from 1¼-inch to ¼-inch with 60 per cent passing the ¾-inch screen. The void content was 36 per cent, dry-rodded, or 1 per cent greater than the A grading. This would seem to be a small difference in voids for so wide a variation in grading. However, repeated tests showed that these values were correct.

Reference to Table 11 will show that the values for the ratio of b to b_0 and for the mortar-voids ratio (volume of mortar as determined by adding the absolute volumes of cement and sand and the volume of water, divided by the volume of voids in the dry-rodded coarse aggregate) were substantially the same for corresponding mixes using A and B grading. Moreover, the values for these ratios in the 1:2:4 mixes using gravel are just about the same as for the 1:2:3½ mixes using crushed stone. On the assumption that these ratios are significant in controlling workability, this would indicate that the workability of the two gravel concretes should be about the same and also that the 1:2:4 gravel concrete should have about the same workability as the 1:2:3½ stone concrete. Referring, however, to Figure 22 and assuming that the presence of honeycomb in the finished structure is a reasonably satisfactory measure of the workability of the concrete going into the structure, we find an average of about 1 per cent honeycomb in the 1:2:3½ stone concrete, about 1½ per cent in the 1:2:4 gravel concrete, grading A, and over 6 per cent in the 1:2:4

TABLE 11.—*Mortar-void ratio and* $\frac{b}{b_0}$ *for all proportions* [1]

Coarse aggregate	Proportions	Average water-cement ratio	Theoretical cement factor	Mortar-voids [2] ratio	$\frac{b}{b_0}$
			Sacks per cubic yard		
Gravel, grading A; b_0=0.65; voids 35 per cent.	1:2:3½	0.86	5.53	2.13	0.717
	1:2:4	.91	5.14	1.89	.762
	1:2:4½	.92	4.84	1.68	.806
	1:2:4¾	.96	4.66	1.63	.820
	1:2:5	.98	4.52	1.55	.838
	1:2:5¼	.97	4.41	1.47	.858
Gravel, grading B; b_0=0.64; voids 36 per cent.	1:2:3½	.89	5.54	2.09	.719
	1:2:4	.92	5.17	1.85	.766
	1:2:4½	.99	4.81	1.69	.802
	1:2:4¾	1.01	4.66	1.61	.820
	1:2:5	1.05	4.51	1.55	.835
	1:2:5¼	1.02	4.41	1.46	.858
Stone, Grading A; b_0=0.60; voids 40 per cent.	1:2:3½	.87	5.73	1.86	.743
	1:2:4	.91	5.35	1.66	.792
	1:2:4¼	.92	5.18	1.56	.816
	1:2:4½	.94	5.02	1.49	.836
	1:2:4¾	.97	4.86	1.43	.854
Stone, grading B; b_0=0.59; voids 41 per cent.	1:2:3½	.87	5.78	1.81	.749
	1:2:4	.91	5.39	1.62	.798
	1:2:4¼	.92	5.22	1.53	.822
	1:2:4½	.95	5.05	1.46	.841
	1:2:4¾	.97	4.90	1.39	.862
Slag, grading A; b_0=0.56; voids 44 per cent.	1:2:3½	.87	5.91	1.69	.766
	1:2:4	.93	5.50	1.52	.815
	1:2:4¼	.95	5.33	1.44	.838
	1:2:4½	.99	5.14	1.38	.857

[1] Compntations were made on the basis of 38 per cent voids for sand and 50 per cent voids (assnmed) for cement.

[2] The mortar-voids ratio for a given mix is obtained by dividing the snm of the absolnte volumes of cement and sand plus the volume of water by the volume of voids in the dry-rodded coarse aggregate. This term should not be confused with the same term as used in the discussion of the mortar-voids theory.

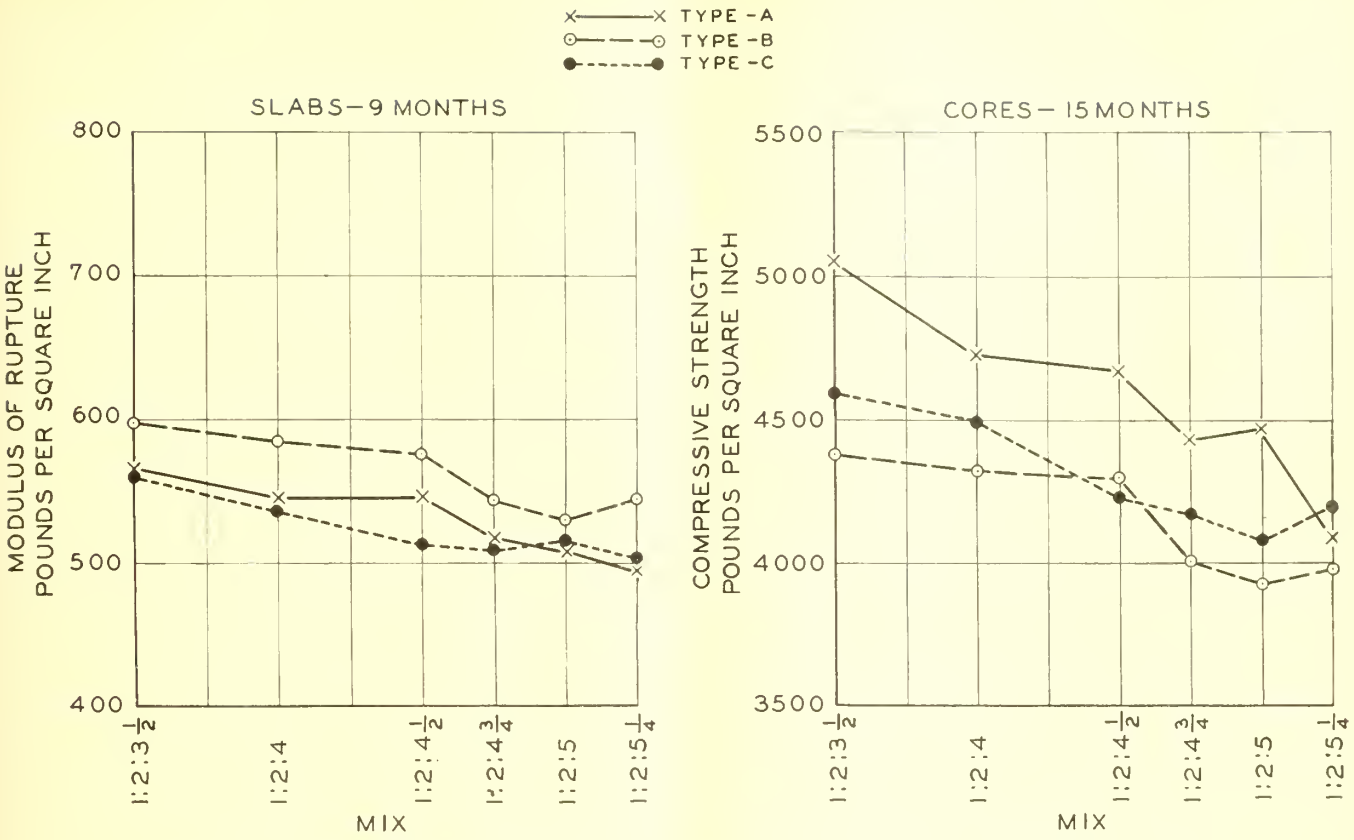


FIGURE 18.—RELATION BETWEEN STRENGTH OF GRAVEL CONCRETE AND TYPE OF FINISHING MACHINE

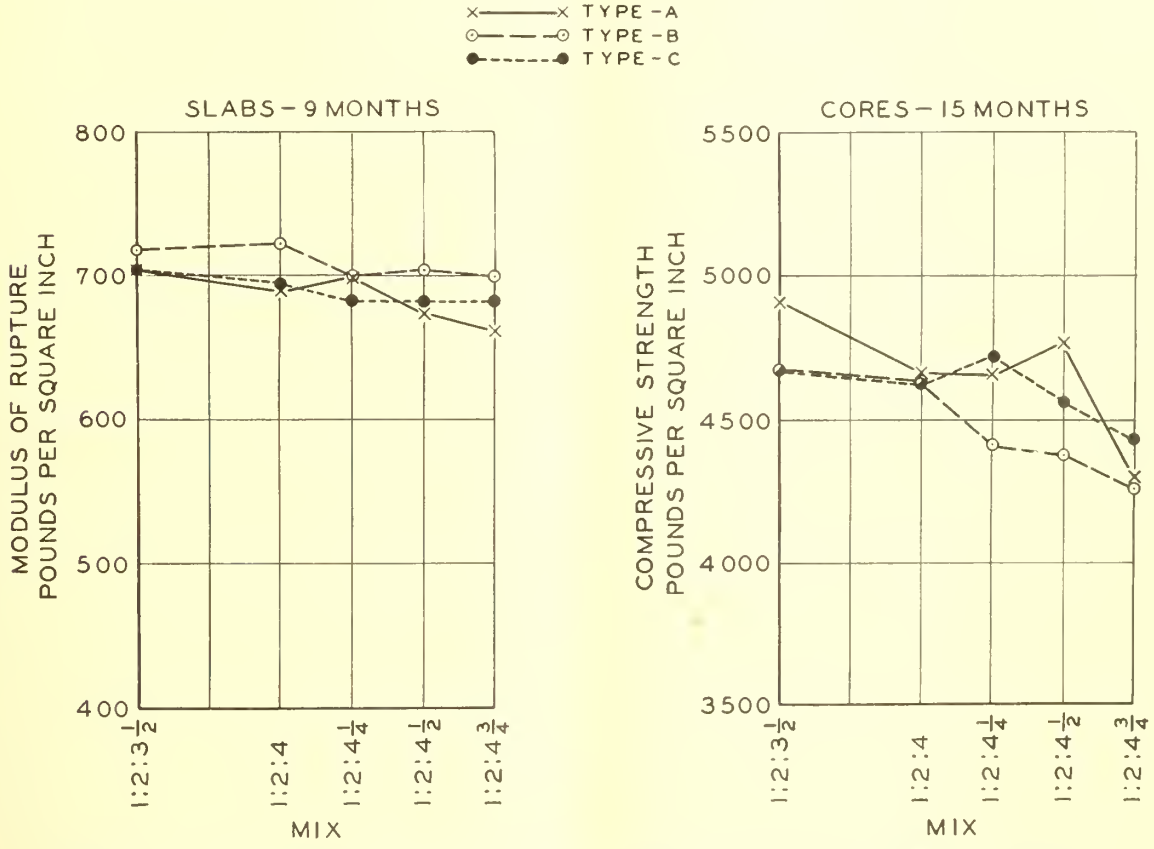


FIGURE 19.—RELATION BETWEEN STRENGTH OF STONE CONCRETE AND TYPE OF FINISHING MACHINE

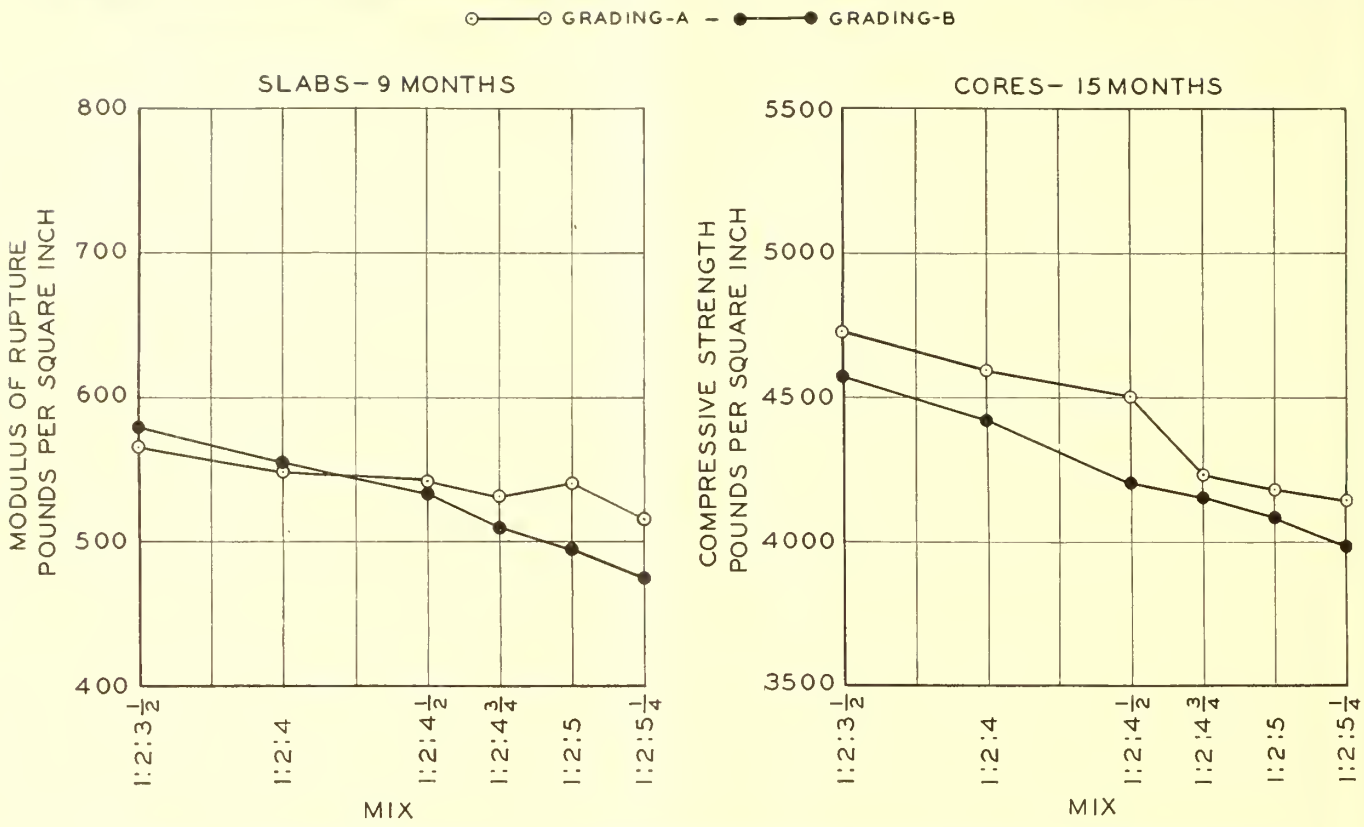


FIGURE 20.—INFLUENCE OF GRADING OF COARSE AGGREGATE AND MIX UPON STRENGTH OF GRAVEL CONCRETE

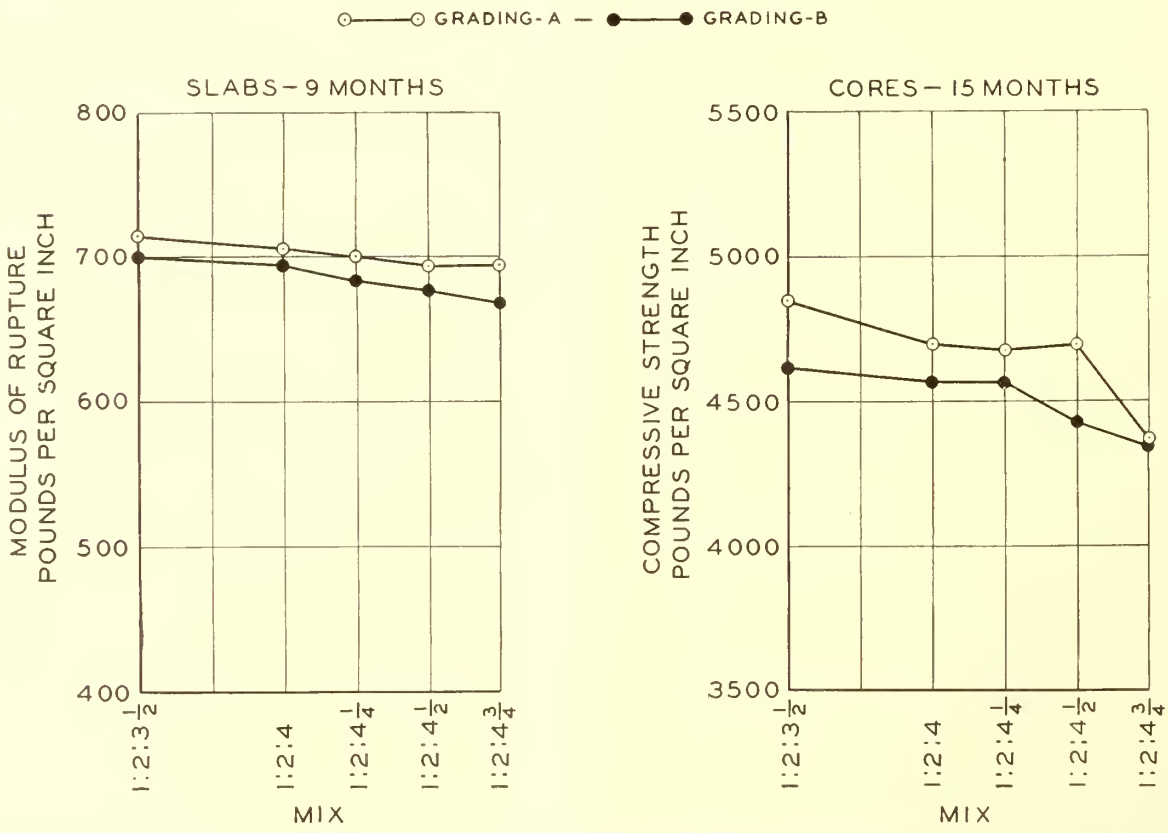


FIGURE 21.—INFLUENCE OF GRADING OF COARSE AGGREGATE AND MIX UPON STRENGTH OF STONE CONCRETE

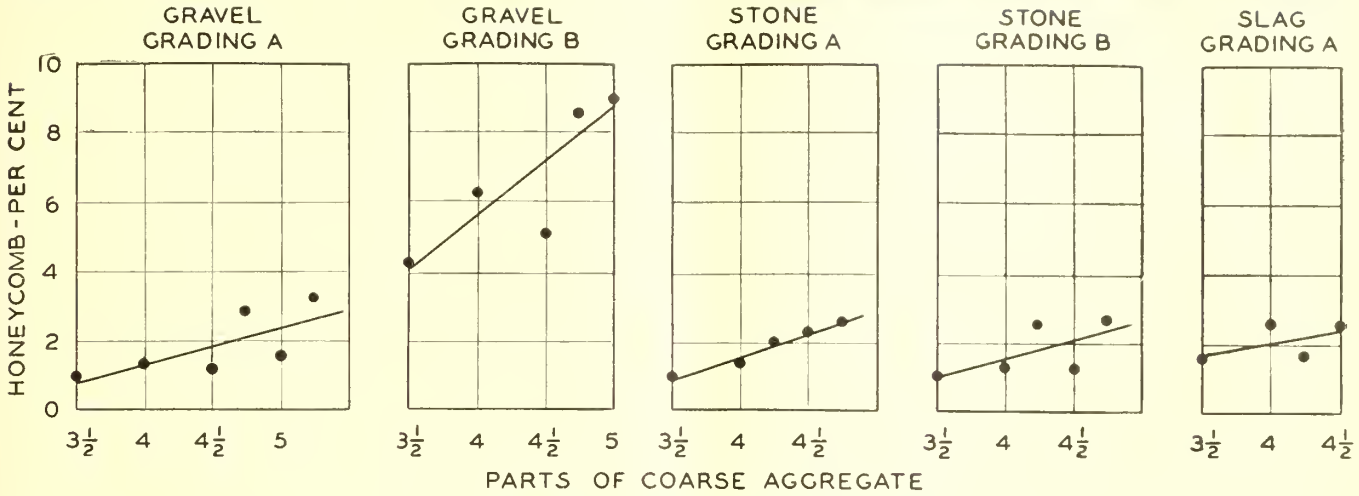


FIGURE 22.—RELATION BETWEEN HONEYCOMB AND MIX

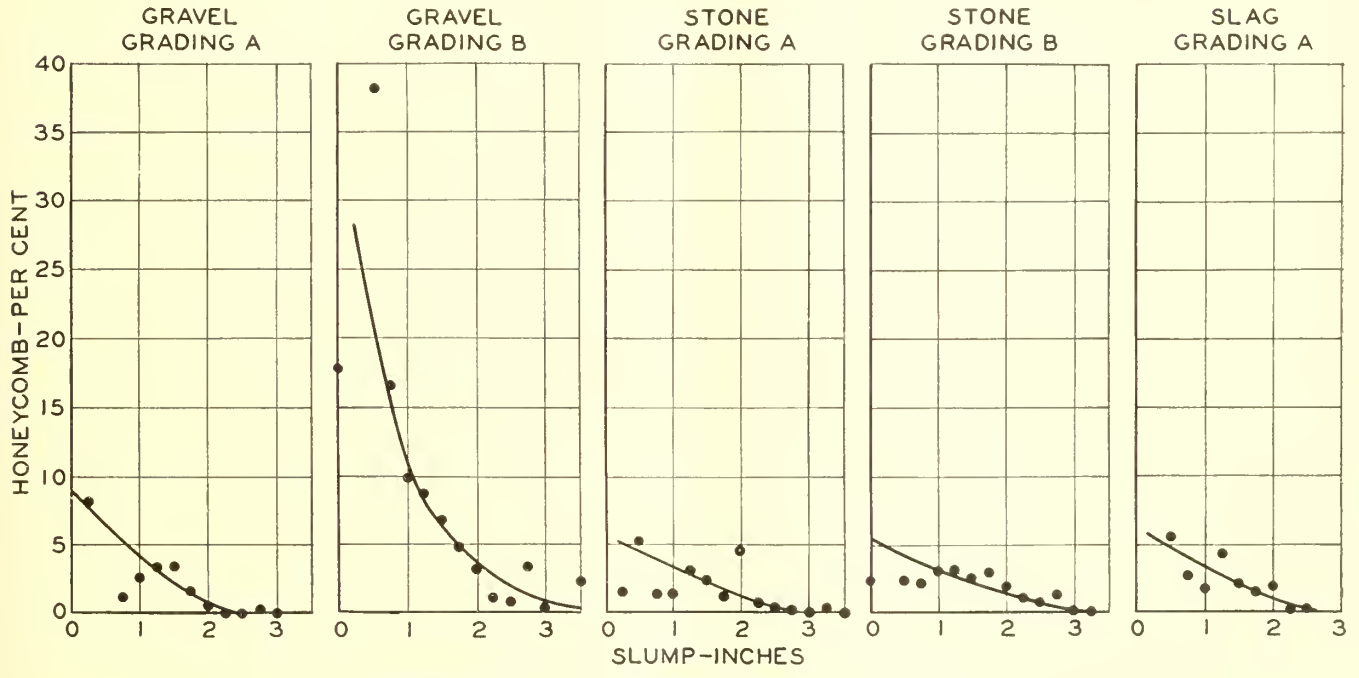


FIGURE 23.—RELATION BETWEEN HONEYCOMB AND SLUMP

gravel concrete, grading B. Even the 1:2:3½ concrete, gravel grading B, showed 4 per cent honeycomb in spite of the fact that this was a distinctly oversanded mix, when analyzed either by mortar voids or fineness modulus theories. In the latter connection it may be noted that, for a 1:2:3½ mix, the volume of fine aggregate is 36 per cent of the sum of the volumes of fine and coarse aggregates. This is more sand than would be called for by the fineness modulus theory for these particular aggregates.

These tests also indicate that the slump test can not be used to compare workability of concretes in which different gradings of coarse aggregate are used. This is illustrated by reference to Figure 23. It will be seen that for a given slump, say one inch, the amount of honeycomb in the case of grading B gravel was much higher than for any other combination.

These data appear also to demonstrate that the use of any function depending directly upon the void content of the coarse aggregate, such as the ratio b/b_0 or the mortar-voids ratio, does not necessarily control the workability of the mix as determined by the uniformity and freedom from honeycomb of the concrete in the structure. They also indicate that, for a given coarse aggregate type and grading, there is a distinct relation between the consistency of the concrete as measured by the slump test and the percentage of honeycomb which is apt to develop under the methods of finishing now in common use.

EFFECT OF HONEYCOMB ON FLEXURAL STRENGTH OF SLABS

It will be of interest now to study the influence of honeycomb upon the strength of the concrete. In Figures 25, 26, and 27, there have been plotted the results of flexure tests on the individual sections arranged by types of finisher and type and gradation of coarse aggregate. Values for strength have been plotted as follows: Open circles designating all slabs having less than 1 per cent honeycomb; crosses, all slabs having 1 to 10 per cent honeycomb; and solid circles, all slabs having more than 10 per cent honeycomb. The three values in each vertical line denote

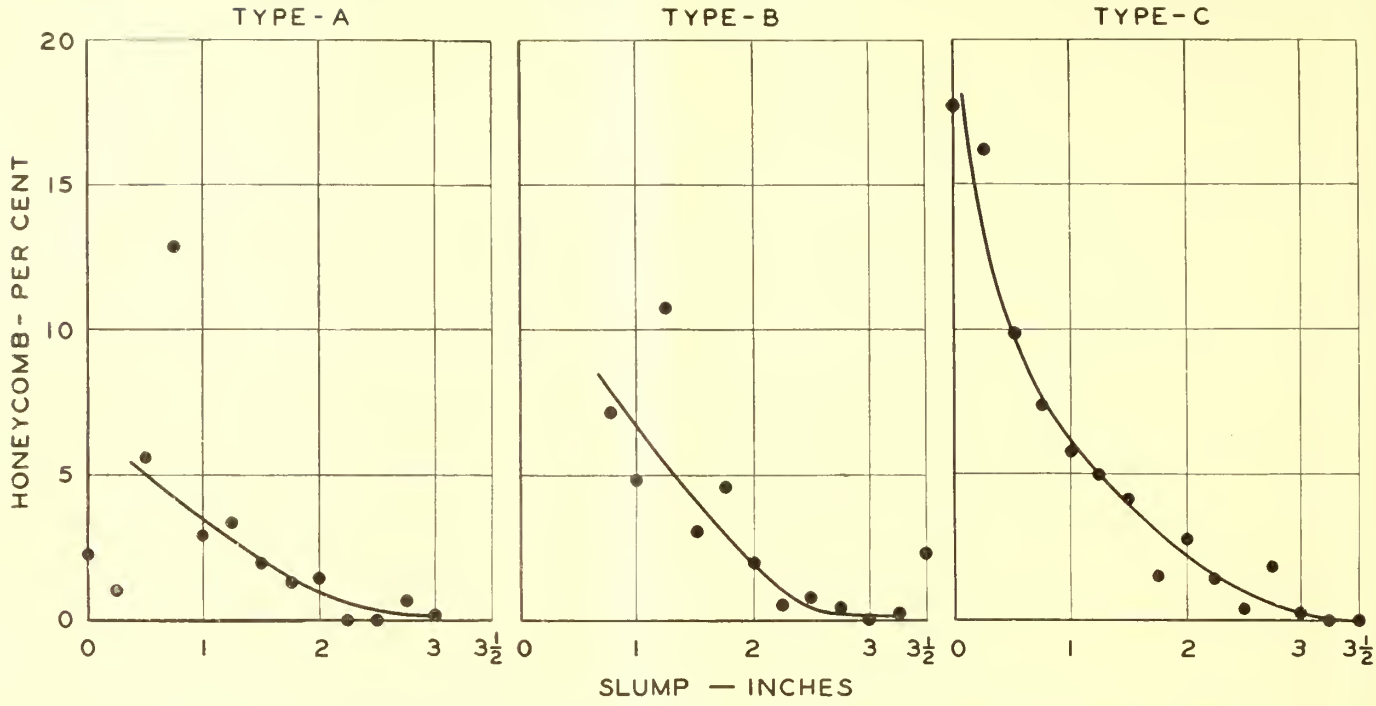


FIGURE 24.—RELATION BETWEEN HONEYCOMB AND SLUMP FOR DIFFERENT TYPES OF FINISHING MACHINE

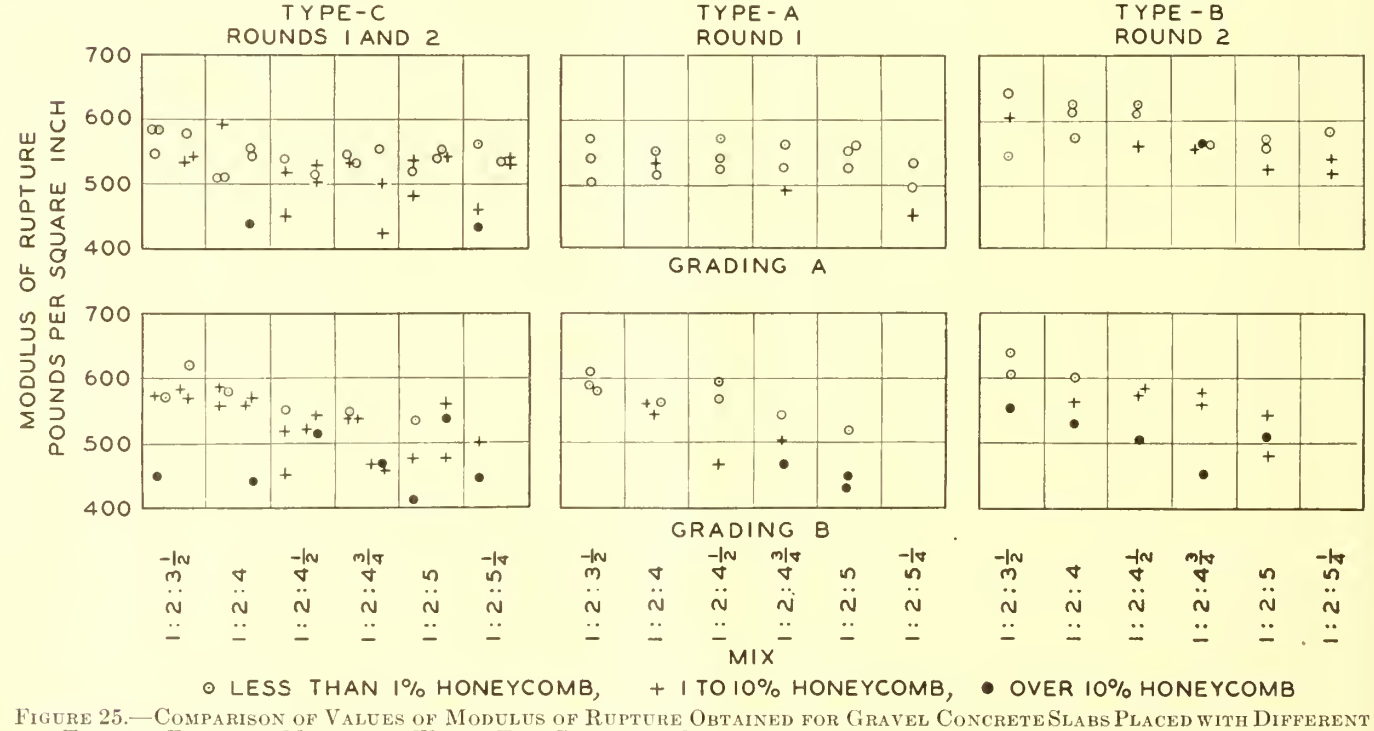

TYPE-C ROUNDS 1 AND 2
TYPE - A ROUND 1
TYPE - B ROUND 2
MODULUS OF RUPTURE POUNDS PER SQUARE INCH
GRADING A
GRADING B
MIX
○ LESS THAN 1% HONEYCOMB, + 1 TO 10% HONEYCOMB, ● OVER 10% HONEYCOMB

FIGURE 25.—COMPARISON OF VALUES OF MODULUS OF RUPTURE OBTAINED FOR GRAVEL CONCRETE SLABS PLACED WITH DIFFERENT TYPES OF FINISHING MACHINE. WHERE TWO GROUPS OF VALUES ARE PLOTTED FOR A GIVEN MIX THOSE ON THE LEFT REPRESENT ROUND 1 AND THOSE ON THE RIGHT REPRESENT ROUND 2

tests on the three sections in which the concrete varied only in consistency.

In order to bring out the interrelated effects of honeycomb, strength, and consistency, these figures should be considered in connection with the detailed results shown in tabulated form in Tables 5, 6, and 13 and in graphic form in Figures 33 to 37, inclusive. Reference to the detailed charts will show a very close relation between honeycomb and consistency in all cases. The concrete in each group of three having the lowest water-cement ratio is almost invariably the concrete having the lowest slump and the largest percentage of honeycomb, if any.

If we refer again to the charts showing relation between honeycomb and strength (figs. 25, 26, and 27), we find that the data indicate conclusively the danger of permitting any condition which will tend to the formation of honeycomb. In the whole group (265 sections) there are 19 sections having more than 10 per cent honeycomb, 15 of which were in the gravel, B grading, group. In all but four cases the badly honeycombed section was distinctly lower in strength than the other

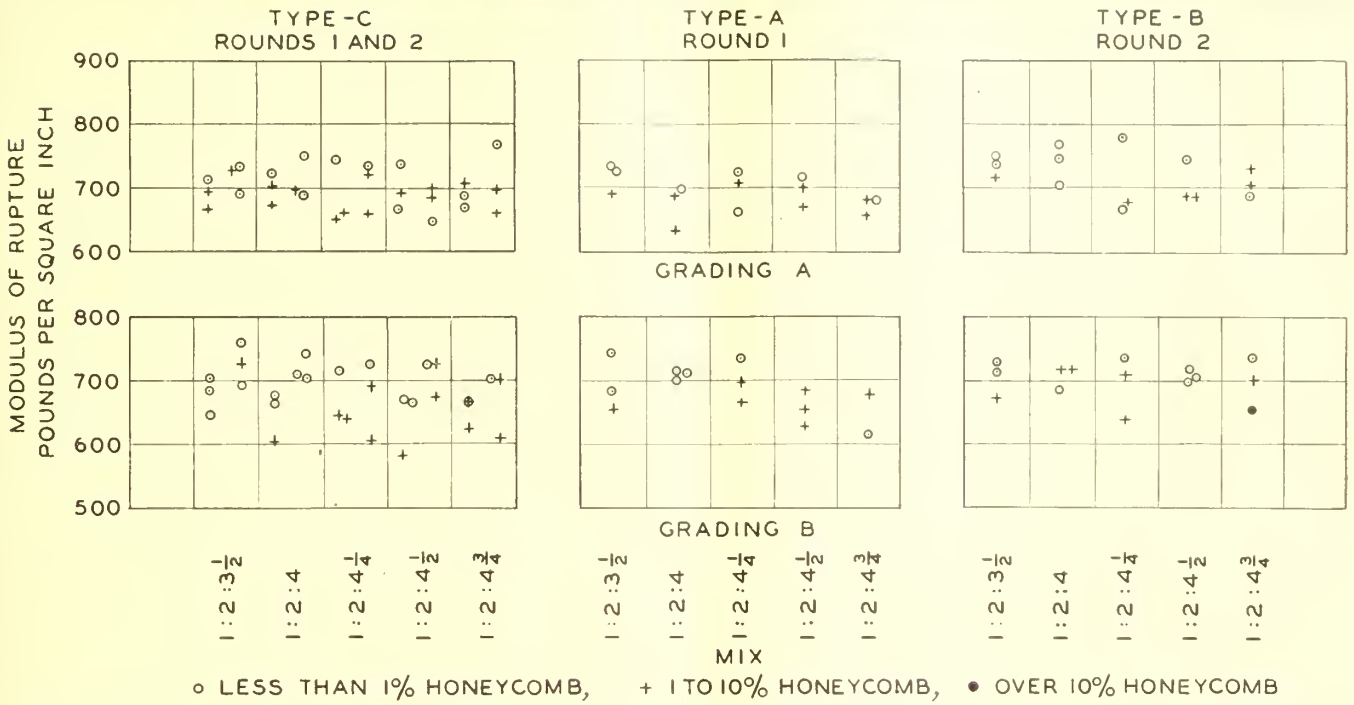


FIGURE 26.—COMPARISON OF VALUES OF MODULUS OF RUPTURE OBTAINED FOR STONE CONCRETE SLABS PLACED WITH DIFFERENT TYPES OF FINISHING MACHINE. WHERE TWO VALUES ARE PLOTTED FOR A GIVEN MIX THOSE ON THE LEFT REPRESENT ROUND 1 AND THOSE ON THE RIGHT REPRESENT ROUND 2

sections of the same group. The same relation exists for the slabs showing from 1 to 10 per cent honeycomb. There were 119 such sections. In only 26 of these was the strength of the honeycombed slab as high or higher than any section in the group showing less than 1 per cent honeycomb. These results emphasize again the danger of using very dry concrete with our present methods of finishing.

A number of interesting detail comparisons may be made by reference to Figures 25, 26, and 27. For instance, it will be noted that there was almost a complete absence of honeycomb in the sections containing gravel, grading A, and finished with type A machine. (Fig. 25.) The corresponding group using stone had a considerably larger number of slabs showing from 1 to 10 per cent honeycomb. Concrete, gravel, B grading, also showed somewhat less honeycombing when finished with type A machine than with either of the other types, although, as has previously been pointed out, the amount of honeycombing in general was much greater for grading B than for grading A. (Fig. 25.) The type of finisher appeared to make little difference in the case of the stone concrete, sections finished with each type showing about the same amount of honeycomb.

The average strength level on the other hand is somewhat higher for type B finisher than for the other types.

In order to form some idea as to the effect of honeycombing on the relation between the strengths of the slabs and the strength of control beams tested at the same age, Figures 28, 29, and 30 have been prepared. These figures indicate for each of the three degrees of honeycombing and for each type and grading of aggregate, the average modulus of rupture for the slabs and the average modulus of rupture for the corresponding beams. It will be understood that each point in this case represents the average strength of all slabs having the amount of honeycomb within the range indicated, with the corresponding values for the beams representing the average strength of the control beams corresponding to those slabs.

These data indicate, in general, the tendency of the slab strengths to decrease with respect to the strengths

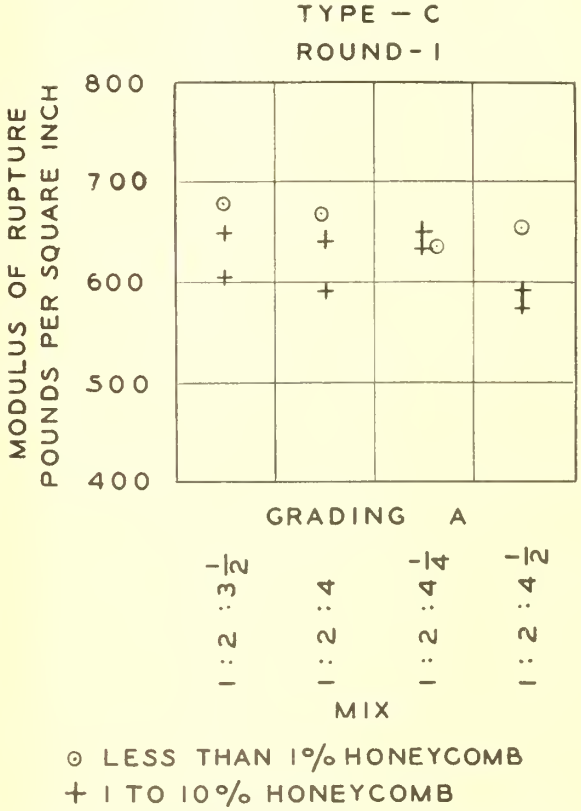


FIGURE 27.—VALUES OF MODULUS OF RUPTURE OBTAINED FOR SLAG CONCRETE SLABS PLACED WITH TYPE C FINISHING MACHINE

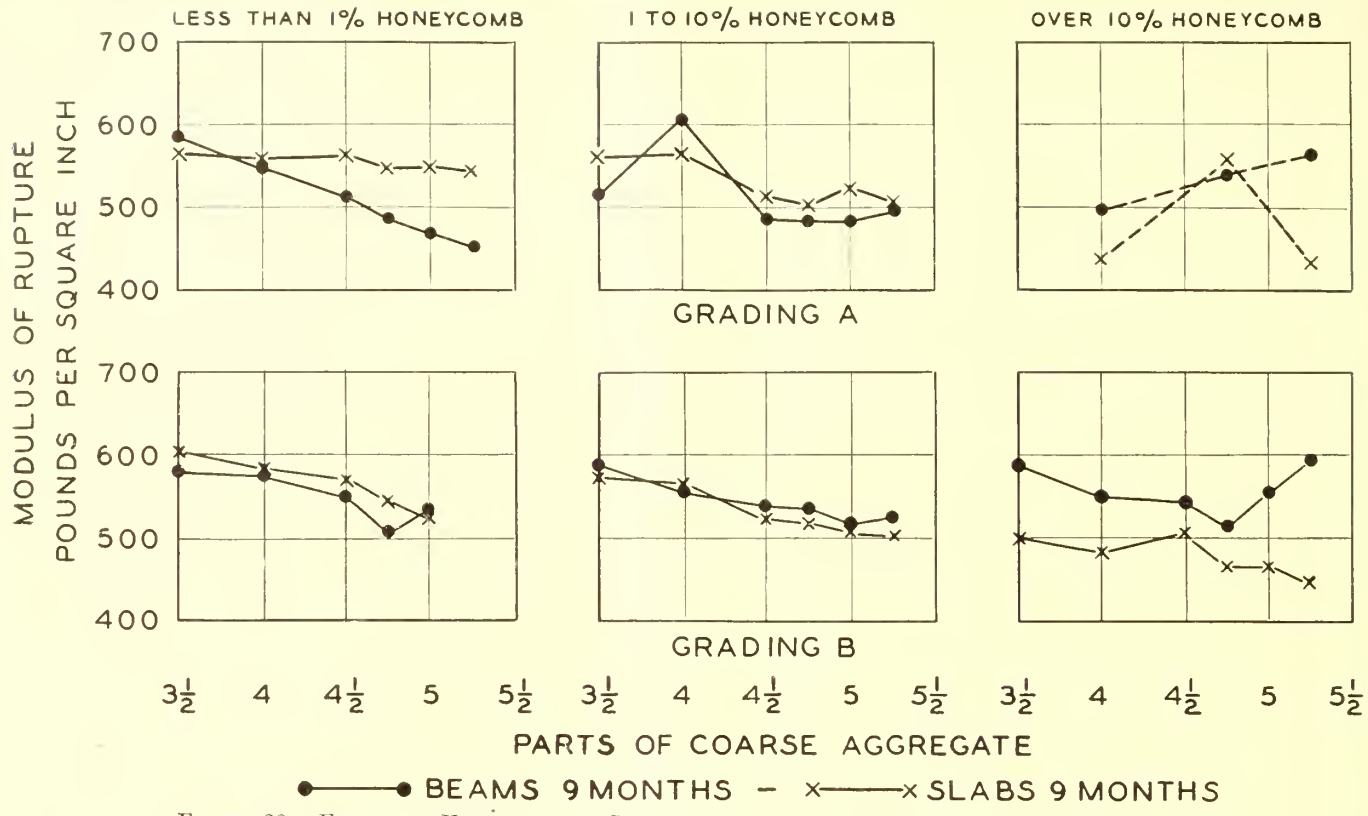


FIGURE 28.—EFFECT ON HONEYCOMB IN SLABS ON MODULUS OF RUPTURE OF GRAVEL CONCRETE

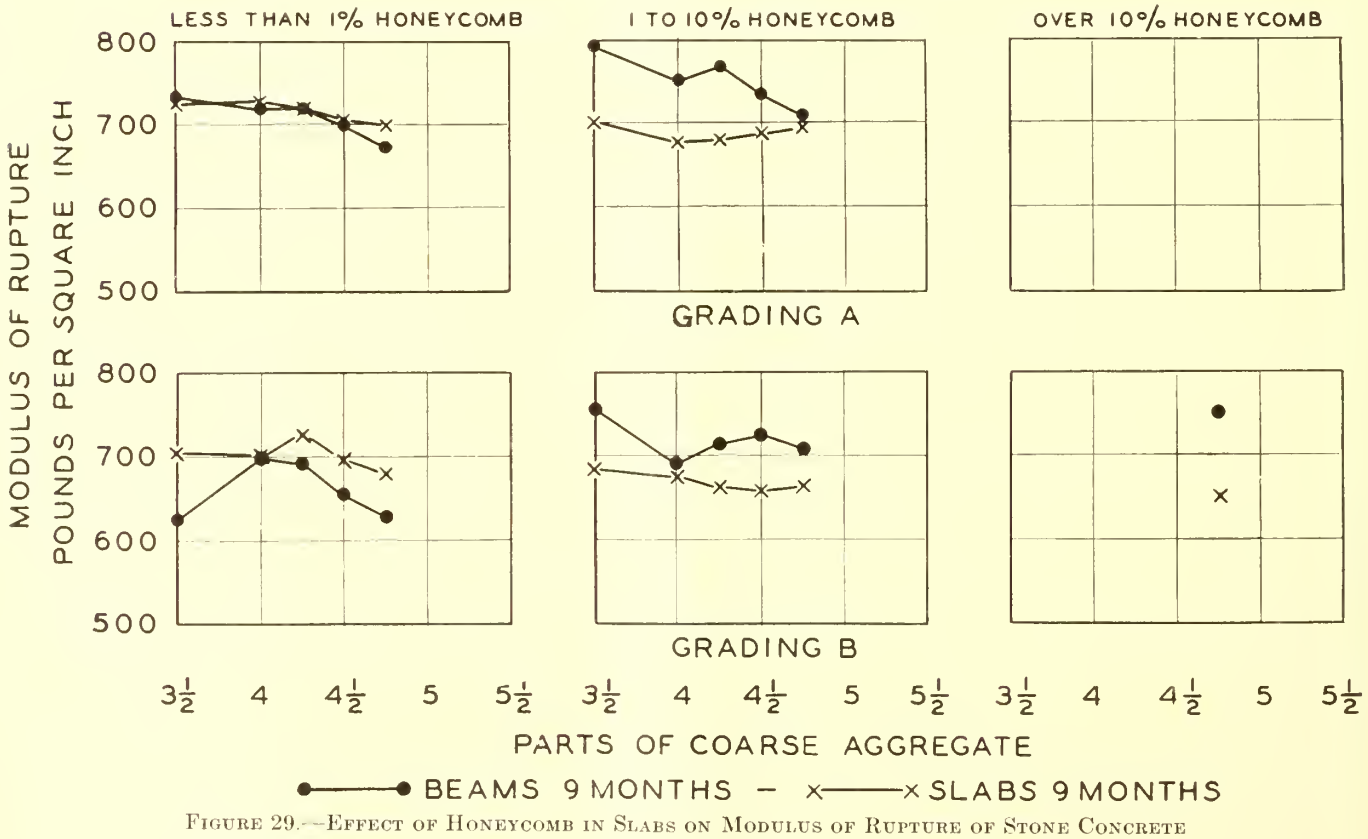


FIGURE 29.—EFFECT OF HONEYCOMB IN SLABS ON MODULUS OF RUPTURE OF STONE CONCRETE

of the control beams for increasing percentages of honeycomb. There are a number of individual values out of line, but the trend is easily followed. It is interesting to observe that in practically all cases where the slabs show less than 1 per cent honeycomb the average slab strengths are as high as or higher than the beam strengths, whereas the reverse is generally true for the slabs showing more than 1 per cent honeycomb.

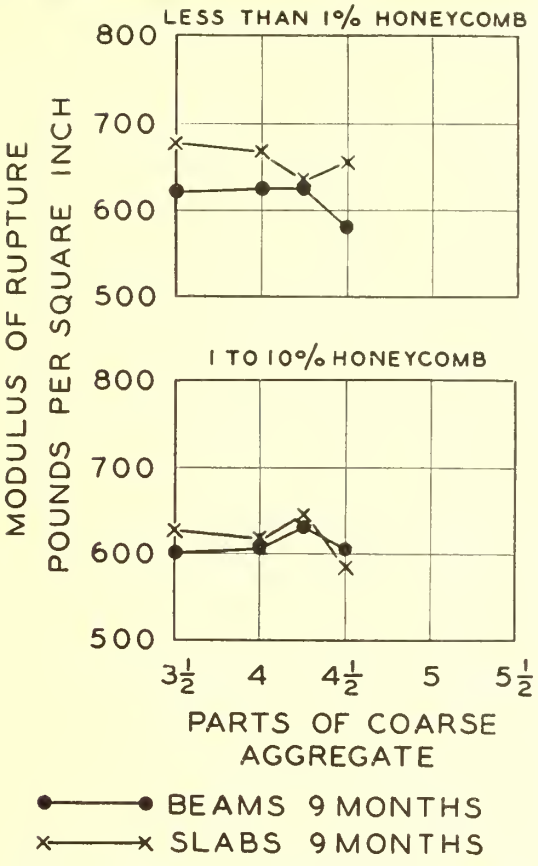


FIGURE 30.—EFFECT OF HONEYCOMB IN SLABS ON MODULUS OF RUPTURE OF SLAG CONCRETE

TABLE 12.—*Notes on finish and workability*

Section No.	Type of finisher	Number passes of screed: Front	Number passes of screed: Rear	Number of tamps	Finish	Workability	Remarks
1	A	3		1	Good	Good	
2	A	4		2	Good	Fair	
3	A	3		2	Good	do	Maximum workable stiffness.
4	A	3		2	Very good	Good	
5	A	4		3	Good	Fair	Some patching.
6	A				Very good	Good	
7	A	2		2	Good	do	
8	A	4		3	Fair	Fair	Maximum workable stiffness; some patching.
9	A	2		1	Good	Good	
10	A	3		2	do	do	
11	A	3		1	do	do	
12	A	4		1	do	do	
13	A	3		2	do	do	
14	A	3		0	do	do	
15	A	4		0	do	do	
16	A	3		0	do	do	
17	A	3		1	do	do	
18	A	4		0	do	do	
19	A	3		1	do	do	Stiff consisteney.
20	A	3		1	do	do	
21	A	3		1	do	Very good	
22	A	3		2	do	Good	
23	A	3		2	do	Very good	
24	A	3		2	do	do	
25	A	3		2	do	Good	
26	A	3		2	do	do	
27	A	3		1	do	do	
28	A	2		1	do	do	
29	A	3		1	do	Fair	Somewbat harsh.
30	A	3		1	do	Good	
31	A	3		1	do	do	Do.
32	A	3		3	Fair	Fair	Difficult to handle.
33	A	3		2	Good	do	
34	C	3	3		Fair	Good	
35	C	2	4		Good	Very good	
36	C	2	3		do	do	

TABLE 12.—*Notes on finish and workability*—Continued

Section No.	Type of finisher	Number passes of screed: Front	Number passes of screed: Rear	Number of tamps	Finish	Workability	Remarks
37	C	2	3		Fair	Good	C. A. rolled under belt.
38	C	3	3		do	do	
39	C	2	3		do	do	
40	C	3	3		do	do	Do.
41	C	2	3		do	Fair	Do.
42	C	2	3		Good	Good	
43	C	2	5		Fair	Fair	Some patching.
44	C	2	2		Good	Very good	
45	C	2	2		do	do	
46	C	6	4		do	Fair	Sligbt segregation.
47	C	3	3		do	Good	
48	C	3	3		do	do	
49	C	3	4		do	Fair	Harsb, some segregation.
50	C	3	3		Fair	Poor	Dry, rather barsh.
51	C	2	2		Good	Good	
52	C	3	4		do	Fair	
53	C	2	2		do	Good	
54	C	2	3		do	do	
55	C	3	3		do	do	
56	C	3	3		do	Very good	
57	C	2	3		do	Good	
58	C	2	2		do	do	
59	C	3	4		do	do	
60	C	3	2		do	do	
61	C	4	3		do	Fair	Some patcbing.
62	C	2	3		do	Good	
63	C	3	3		do	do	Little mortar in front of screed.
64	C	2	4		Fair	do	C. A. rolled under belt.
65	C	3	3		do	Fair	Harsh.
66	C	2	3		do	Good	C. A. rolled under belt.
67	C	2	4		do	Fair	Harsh.
68	C	3	3		Very poor	Unworkable	Very harsh, segregated.
69	A	3		1	Good	Very good	
70	A	3		1	Fair	Good	C. A. rolled under belt.
71	A	5		4	Poor	Very poor	Part of surface rough.
72	A	3		1	Fair	Good	C. A. rolled under belt.
73	A	2		0	do	Very good	Do.
74	A	3		1	do	Good	Do.
75	A	3		2	do	do	C. A. rolled under screed and belt.
76	A	3		1	do	Very good	Too dry wben belted.
77	A	3		2	Rough	Poor	Very dry.
78	A	3		3	do	Fair	Very dry, some patcbing.
79	A	3		1	Fair	Good	C. A. rolled under belt.
80	A	3		1	Good	do	
81	A	3		3	Poor	Unworkable	
82	A	3		2	Fair	Fair	Some patching.
83	A	3		1	Good	Good	
84	A	4		2	Fair	Very good	Belted too dry.
85	A	3		2	do	Fair	Some patching.
86	A	3		1	Good	Very good	
87	A	3		2	Rough	Fair	Do.
88	A	4		1	Fair	Good	
89	A	4		1	Good	do	
90	A	4		3	Fair	Fair	Somewbat barsh.
91	A	3		2	Good	Good	
92	A	3		1	do	do	
93	A	3		2	Fair	Fair	
94	A	3		1	Good	do	Do.
95	A	3		1	do	Good	
96	A	3		3	Rough	Poor	Rough patches.
97	A	3		0	Good	Good	Rather wet.
98	C	3	3		do	do	
99	C	3	2		do	do	
100	C	3	3		Fair	Fair	Some patching.
101	C	2	3		do	do	
102	C	2	2		Good	Very good	
103	C	2	3		do	do	
104	C	3	3		Fair	Fair	C. A. rolled under screed and belt.
105	C	2	3		Good	Good	
106	C	2	3		Fair	Fair	
107	C	2	3		do	do	
108	C	2	3		Good	Good	
109	C	2	3		do	do	
110	C	3	3		Fair	Fair	Somewbat barsh.
111	C	2	3		Good	Good	
112	C	2	3		do	do	
113	C	2	3		do	Fair	
114	C	3	3		do	Good	
115	C	2	3		do	do	
116	C	2	3		do	do	
117	C	2	3		do	Fair	
118	C	2	3		do	Good	
119	C	2	2		do	do	
120	C	2	2		do	do	
121	C	3	3		Fair	Poor	
122	C	2	3		do	Fair	Some patcbing.
123	C	2	3		Good	Good	
124	C	2	2		do	do	
125	C	3	3		do	do	Some segregation.
126	C	3	3		do	Fair	Do.
127	C	2	3		do	do	Do.

TABLE 12.—*Notes on finish and workability*—Continued

Section No.	Type of finisher	Number passes of screed: Front	Number passes of screed: Rear	Number of tamps	Finish	Workability	Remarks
128	C	2	2		Good	Good	
129	C	2	2		Fair	Fair	
130	C	2	2		Good	Good	
131	C	2	2		do	do	
132	C	2	2		do	do	
133	C	2	3		do	Fair	
134	C	2	3		Fair	do	
135	C	2	2		Good	Good	
136	C	2	2		do	do	
137	C	2	3		do	Fair	
138	C	2	2		do	Good	
139	C	2	2		do	do	
140	C	3	4		Rough	Fair	
141	C	2	2		Good	Good	
142	C	2	2		do	do	
143	C	2	2		do	Fair	
144	C	2	2		do	do	
145	C	2	2		do	Good	
146	C	2	3		Poor	Fair	Too dry.
147	C	2	2		Good	Good	
148	C	2	3		do	do	
149	C	2	3		Fair	Fair	
150	C	2	2		Good	do	
151	C	2	2		do	Good	
152	C	2	2		do	do	
153	C	2	2		do	do	
154	C	2	2		do	do	
155	C	3	3		Very poor	Poor	Unworkable in spots.
156	C	2	2		Good	Good	
157	C	2	2		do	do	
158	C	3	3		Poor	Poor	
159	C	2	3		Good	Fair	
160	C	2	2		do	Good	
161	B	4			Fair	Fair	C. A. rolled under belt.
162	B	4			do	Good	Do.
163	B	3			Good	do	
164	B	3			Fair	do	Do.
165	B	3			do	do	Do.
166	B	3			do	Fair	
167	B	5			do	do	Harsh.
168	B	3			do	do	
169	B	3			Good	Good	
170	B	3			Poor	Poor	Hand finishing necessary.
171	B	3			Good	Good	
172	B	3			do	do	
173	B	3			do	do	
174	B	3			Fair	do	Some segregation.
175	B	3			Poor	Poor	Hand finishing necessary.
176	B	3			Good	Good	Some segregation.
177	B	3			do	do	
178	B	4			Poor	Poor	Harsh.
179	B	3			Good	Good	
180	B	3			Fair	Fair	
181	B	3			Good	Good	
182	B	3			do	do	
183	B	3			do	do	
184	B	4			Rough	Poor	Hand finishing necessary.
185	B	3			Good	Good	
186	B	3			do	do	
187	B	3			do	Fair	
188	B	3			do	Very good	
189	B	3			do	Fair	
190	B	3			do	do	
191	B	3			do	Very good	
192	B	3			do	Good	
193	B	3			do	Fair	
194	B	4			Rough	Unworkable	
195	B	3			Good	Good	
196	B	3			do	do	
197	B	3			do	do	
198	B	2			Fair	Fair	Two extra screeds on half of section.
199	B	3			Good	Good	
200	B	3			do	do	
201	B	3			do	do	
202	B	3			do	do	
203	B	3			do	do	
204	B	3			do	Fair	
205	B	3			Rough	Unworkable	
206	B	3			Good	Good	
207	B	3			do	do	
208	B	4			Rough	Fair	Some patching.
209	B	3			Good	do	
210	B	3			Fair	Poor	
211	B	4			Poor	Unworkable	Patching necessary.
212	B	3			Good	Very good	
213	B	3			do	Good	
214	B	4			Fair	Fair	Do.
215	B	3			Good	Good	
216	B	3			do	do	
217	B	3			Poor	Unworkable	Third of surface hand finished.
218	B	3			Good	Good	Harsh, wet, some segregation.
219	B	3			do	do	Do.
220	B	3			Poor	Poor	Patching due to segregation.
221	B	3			Good	Good	Harsh, wet, some segregation.
222	B	3			do	Fair	Patching due to segregation.
223	B	4			Rough	Very poor	Do.
224	C	2	2		Good	Very good	
225	C	2	2		do	do	
226	C	2	2		do	Fair	
227	C	2	2		do	Good	
228	C	2	2		do	do	
229	C	2	2		Fair	Fair	
230	C	2	3		do	do	
231	C	2	2		Good	Good	
232	C	2	2		do	Very good	
233	C	2	2		do	Good	
234	C	2	2		do	do	
235	C	2	2		do	do	
236	C	2	2		do	Fair	Rather harsh.
237	C	2	2		do	Good	
238	C	2	2		do	Very good	
239	C	2	2		Fair	Fair	
240	C	2	2		Good	Good	
241	C	2	2		do	Very good	
242	C	2	2		do	Fair	
243	C	2	2		do	Good	
244	C	2	2		do	do	
245	C	2	2		do	Fair	
246	C	2	2		do	Good	
247	C	2	3		Rough	Poor	
248	C	2	2		Good	Good	
249	C	2	2		do	Fair	
250	C	2	2		do	Good	
251	C	2	2		do	Fair	
252	C	2	2		do	do	
253	C	2	2		do	do	
254	C	2	2		do	do	
255	C	2	2		do	Good	
256	C	2	2		Rough	Poor	Hand finishing necessary.
257	C	2	2		Poor	Very poor	Do.
258	C	2	2		Good	Fair	Some patching.
259	C	2	2		do	Good	
260	C	2	2		Fair	Fair	
261	C	2	2		Good	Good	
262	C	2	2		Fair	Poor	Patching necessary.
263	C	2	2		Good	Good	
264	C	2	2		do	do	
265	C	2	2		Fair	Poor	Do.

UNIFORMITY OF CONCRETE ANALYZED

It will be recalled that the value for the modulus of rupture of each test slab was obtained by averaging the results of tests on the four slabs which made up the section. A study of the variation in strength among the four slabs should give some idea, therefore, of the uniformity of the concrete within the section.

Table 13 gives for each test section the results of flexure tests on the individual slabs, together with the percentage variation of each slab from the average for the section and the average percentage variation for the section. This latter figure may be used as the index of uniformity for the section. In Figure 31 the average percentage variation in strength for all of the gravel concrete sections and all of the stone concrete sections for each proportion have been plotted. It will be seen that there is a reasonably consistent reduction in uniformity with increase in coarse aggregate content for both types of coarse aggregate. The average deviation varies from about 4 per cent to about 7 per cent for the entire range. By breaking these average values up into group averages corresponding to grading of aggregate, type of finisher, etc., similar results are obtained. The difference between the effects of A and B grading in

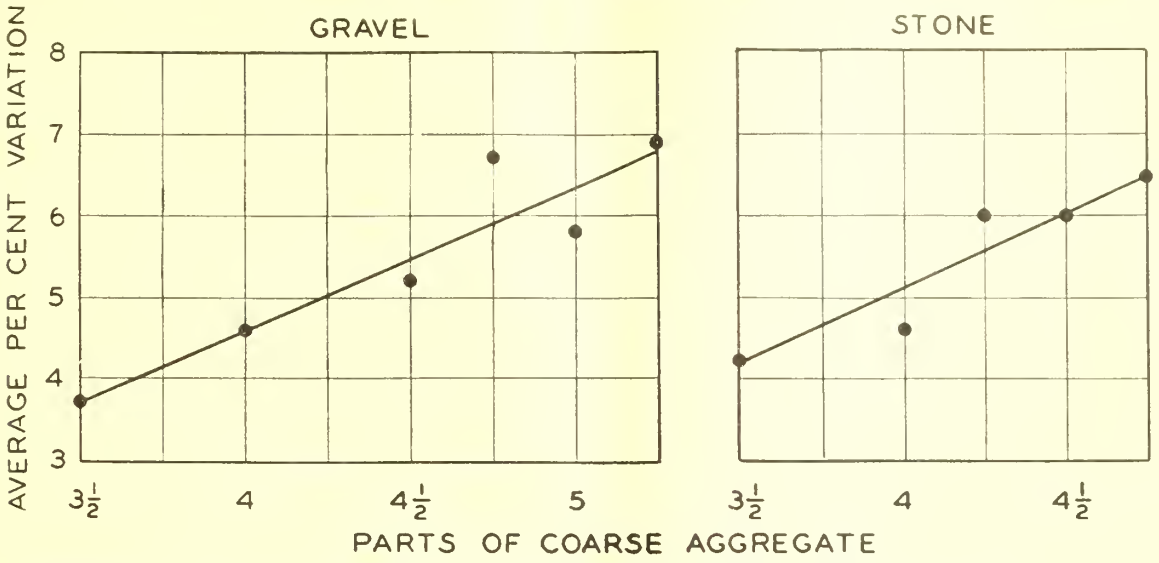


FIGURE 31.—UNIFORMITY OF CONCRETE: AVERAGE PERCENTAGE VARIATION OF MODULUS OF RUPTURE OF INDIVIDUAL SLABS FROM AVERAGE MODULUS OF RUPTURE FOR THE SECTION OF 4 SLABS

the gravel which is so marked as regards honeycombing does not appear from the standpoint of uniformity. Neither can any consistent difference in uniformity be observed as the result of using the different methods of finishing. Each method gives about the same average degree of uniformity.

However, study of Table 13 indicates some relation between honeycombing and uniformity. For instance, the average deviation for the 19 sections showing more than 10 per cent honeycomb is 9.2 per cent against a grand average of 5.4 for the entire series. Conversely, the 127 sections having less than 1 per cent honeycomb showed an average deviation of only 3.8 per cent. These values are given to illustrate trends only as there are several cases where honeycombed slabs show good uniformity and vice versa. The trend, however, is in the direction which would naturally be expected and further illustrates the importance of guarding against danger of honeycombing in the construction of concrete pavements.

TABLE 13.—*Variation in modulus of rupture, of individual slabs from average for the section of 4 slabs, and percentage of honeycomb*

Section No.	Modulus of rupture in pounds per square inch					Percentage variation from average					Honeycomb
	1	2	3	4	Average	1	2	3	4	Average	
											Per cent
1	539	545	552	531	542	0.6	0.6	1.8	2.0	1.2	0.1
2	586	562	535	608	573	2.3	1.9	6.6	6.1	4.2	0
3	508	498	514	497	504	.8	1.2	2.0	1.4	1.4	.2
4	532	582	523	577	554	4.0	5.1	5.6	4.2	4.7	0
5	429	457	538	702	532	19.4	14.1	1.1	32.0	16.6	1.3
6	481	547	511	540	520	7.5	5.2	1.7	3.8	4.6	.6
7	531	493	558	535	529	.4	6.8	5.5	1.1	3.4	0
8	587	574	552	585	575	2.1	.2	4.0	1.7	2.0	.6
9	556	534	509	576	544	2.2	1.8	6.4	5.9	4.1	.2
10	542	500	542	537	530	2.3	5.7	2.3	1.3	2.9	0
11	561	512	586	597	564	.5	9.2	3.9	5.9	4.9	0
12	580	398	487	523	497	16.7	19.9	2.0	5.2	11.0	3.8
13	515	501	589	624	557	7.5	10.1	5.7	12.0	8.8	.1
14	500	562	525	530	529	5.5	6.2	.8	.2	3.2	0
15	556	539	565	587	562	1.1	4.1	.5	4.4	2.5	0
16	483	505	500	511	500	3.4	1.0	0	2.2	1.6	0
17	447	421	488	457	453	1.3	7.1	7.7	.9	4.2	5.2
18	506	539	542	553	535	5.4	.7	1.3	3.4	2.7	0
19	611	565	613	647	609	.3	7.2	.7	6.2	3.6	.9
20	627	570	586	578	590	6.3	3.4	.7	2.0	3.1	0
21	599	548	572	602	580	3.3	5.5	1.4	3.8	3.5	0
22	557	529	582	499	542	2.8	2.4	7.4	7.9	5.1	2.9
23	558	569	560	547	559	.2	1.8	.2	2.1	1.1	3.2

TABLE 13.—*Variation in modulus of rupture, of individual slabs from average for the section of 4 slabs, and percentage of honeycomb*—Continued

Section No.	Modulus of rupture in pounds per square inch					Percentage variation from average					Honeycomb
	1	2	3	4	Average	1	2	3	4	Average	
											Per cent
24	547	546	577	575	561	2.5	2.7	2.9	2.5	2.6	0
25	471	488	459	448	466	1.1	4.7	1.5	3.9	2.8	6.3
26	549	580	651	591	593	7.4	2.2	9.8	.3	4.9	.6
27	588	572	547	557	566	3.9	1.1	3.4	1.6	2.5	.6
28	483	571	563	547	541	10.7	5.5	4.1	1.1	5.4	.2
29	443	532	511	525	503	11.9	5.8	1.6	4.4	5.9	5.8
30	509	491	415	455	468	8.8	4.9	11.3	2.8	7.0	19.2
31	498	586	499	494	519	4.0	12.9	3.9	4.8	6.4	.6
32	442	409	431	439	430	2.8	4.9	.2	2.1	2.5	25.4
33	476	481	470	362	447	6.5	7.6	5.1	19.0	9.6	12.9
34	569	591	600	573	583	2.4	1.4	2.9	1.7	2.1	0
35	502	517	554	613	547	8.2	5.5	1.3	12.1	6.8	0
36	567	569	600	596	583	2.7	2.4	2.9	2.2	2.6	0
37	626	552	592	607	594	5.4	7.1	.3	2.2	3.8	1.2
38	484	567	494	500	511	5.3	11.0	3.3	2.2	5.4	.5
39	527	500	483	520	508	3.7	1.6	4.9	2.4	3.2	.1
40	510	387	393	516	452	12.8	14.4	13.1	14.2	13.6	2.1
41	535	517	451	570	518	3.3	.2	12.9	10.0	6.6	1.0
42	512	545	547	557	540	5.2	.9	1.3	3.1	2.6	0
43	521	603	583	441	537	3.0	12.3	8.6	17.9	10.4	4.3
44	541	552	510	536	535	1.1	3.2	4.7	.2	2.3	0
45	570	554	458	588	543	5.0	2.0	15.7	8.3	7.8	0
46	548	540	536	525	537	2.0	.6	.2	2.2	1.2	3.9
47	504	501	485	441	483	4.3	3.7	.4	8.7	4.3	2.8
48	547	492	502	540	520	5.2	5.4	3.5	3.8	4.5	0
49	535	539	361	419	464	15.3	16.2	22.2	9.7	15.8	6.6
50	504	532	281	411	432	16.7	23.1	35.0	4.9	19.9	16.2
51	574	546	544	595	565	1.6	3.4	3.7	5.3	3.5	0
52	531	470	447	339	447	18.8	5.1	0	24.2	12.0	20.5
53	613	565	596	510	571	7.4	1.1	4.4	10.7	5.9	8.2
54	613	558	565	552	572	7.2	2.4	1.2	3.5	3.6	0
55	600	593	605	542	585	2.6	1.4	3.4	7.4	3.7	7.5
56	561	480	576	601	555	1.1	13.5	3.8	8.3	6.7	1.2
57	597	577	585	556	579	3.1	.3	1.0	4.0	2.1	.4
58	500	366	480	459	451	10.9	18.8	6.4	1.8	9.5	9.8
59	579	539	513	435	516	12.2	4.5	.6	15.7	8.2	4.8
60	577	544	516	566	551	4.7	1.3	6.4	2.7	3.8	0
61	565	509	545	528	537	5.2	5.2	1.5	1.7	3.4	6.2
62	539	462	556	587	536	.6	13.8	3.7	9.5	6.9	3.2
63	538	538	564	546	547	1.6	1.6	3.1	.2	1.6	0
64	498	422	268	463	413	20.6	2.2	35.1	12.1	17.5	24.5
65	558	417	438	493	477	17.0	12.6	8.2	3.4	10.3	7.0
66	504	511	539	586	535	5.8	4.5	.7	9.5	5.1	.5
67	512	495	542	454	501	2.2	1.2	8.2	9.4	5.2	4.8
68	410	471	479	432	448	8.5	5.1	6.9	3.6	6.0	17.8
69	762	736	755	677	732	4.1	.5	3.1	7.5	3.8	0
70	715	723	732	730	725	1.4	.3	1.0	.7	.8	0
71	590	729	735	705	690	14.5	5.7	6.5	2.2	7.2	2.2
72	755	648	669	675	687	9.9	5.7	2.6	1.7	5.0	1.2
73	705	705	700	677	697	1.1	1.1	.4	2.9	1.4	0
74	516	664	714	636	632	18.4	5.1	13.0	.6	9.3	8.4
75	790	715	659	736	725	9.0	1.4	9.1	1.5	5.2	.5
76	567	685	746	643	660	14.1	3.8	13.0	2.6	8.4	0
77	683	687	724	739	708	3.5	3.0	2.3	4.4	3.3	2.1
78	721	559	780	621	670	7.6	16.6	16.4	7.3	12.0	6.5
79	739	730	655	675	700	5.6	4.3	6.4	3.6	5.0	2

TABLE 13. *Variation in modulus of rupture, of individual slabs from average for the section of 4 slabs, and percentage of honeycomb*—Continued

Section No.	Modulus of rupture in pounds per square inch					Percentage variation from average					Honeycomb
	1	2	3	4	Average	1	2	3	4	Average	
											Per cent
80	723	705	713	704	711	1.7	.8	.3	1.0	1.0	.3
81	628	712	555	732	657	4.4	8.4	15.5	11.4	9.9	4.5
82	704	577	738	705	681	3.4	15.3	8.4	3.5	7.6	6.2
83	696	661	639	714	678	2.7	2.5	5.8	5.3	4.1	.6
84	817	695	745	709	742	10.1	6.3	.4	4.4	5.3	.5
85	675	634	656	648	653	3.4	2.9	.5	.8	1.9	2.8
86	724	632	700	677	683	6.0	7.5	2.5	.9	4.2	0
87	745	735	653	722	714	4.3	2.9	8.5	1.1	4.2	0
88	643	693	753	719	702	8.4	1.3	7.3	2.4	4.8	0
89	769	657	704	715	711	8.2	7.6	1.0	.5	4.3	0
90	749	596	749	687	695	7.8	14.2	7.8	1.2	7.8	1.6
91	696	616	798	817	732	4.9	15.8	9.0	11.6	10.3	.8
92	709	632	638	685	666	6.5	5.1	4.2	2.9	4.7	1.4
93	757	695	588	697	684	10.7	1.6	14.0	1.9	7.0	2.3
94	693	673	504	642	628	10.4	7.2	19.7	2.2	9.9	1.9
95	671	636	699	611	654	2.6	2.8	6.9	6.6	4.7	1.0
96	562	645	783	722	678	17.1	4.9	15.5	6.5	11.0	4.2
97	623	647	627	556	613	1.6	5.5	2.3	9.3	4.7	.4
98	620	670	631	761	670	7.5	0	5.8	13.6	6.7	1.4
99	673	705	759	721	714	5.7	1.3	6.3	1.0	3.6	0
100	723	696	660	697	694	4.2	.3	.4	4.9	2.4	3.7
101	726	712	629	743	702	3.4	1.4	10.4	5.8	5.2	2.3
102	709	735	714	743	725	2.2	1.4	1.5	2.5	1.9	0
103	721	707	566	711	676	6.7	4.6	16.3	5.2	8.2	1.3
104	664	637	618	690	652	1.8	2.3	5.2	5.8	3.8	1.5
105	792	701	772	722	747	6.0	6.2	3.3	3.3	4.7	.2
106	672	563	703	710	662	1.5	15.0	6.2	7.3	7.5	3.6
107	733	726	622	696	694	5.6	4.6	10.4	.3	5.2	2.0
108	768	694	766	727	739	3.9	6.1	3.7	1.6	3.8	0
109	750	623	657	648	670	11.9	7.0	1.9	3.3	6.0	0
110	722	726	693	705	712	1.4	2.0	2.7	1.0	1.8	3.4
111	735	644	721	659	690	6.5	6.7	4.5	4.5	5.6	.9
112	677	681	655	675	672	.7	1.3	2.5	.4	1.2	0
113		637	697	721	685		7.0	1.8	5.3	4.7	.6
114	721	691	718	683	703	2.6	1.7	2.1	2.8	2.3	0
115	653	647	631	646	644	1.4	.5	2.0	.3	1.0	0
116	679	688	609	682	664	2.3	3.6	8.3	2.7	4.2	0
117	689	597	573	551	602	14.5	.8	4.8	8.5	7.2	9.5
118	691	659	670	685	676	2.2	2.5	.9	1.3	1.7	0
119	747	711	668	725	713	4.8	.3	6.3	1.7	3.3	0
120	751	498	672	658	645	16.4	22.8	4.2	2.0	11.4	2.6
121	549	691	692	626	640	14.2	8.0	8.1	2.2	8.1	3.8
122	458	657	588	630	583	21.4	12.7	.9	8.1	10.8	3.8
123	639	693	675	673	670	4.6	3.4	.7	.4	2.3	.6
124	711	619	674	659	666	6.8	7.0	1.2	1.0	4.0	0
125	632	617	616	634	625	1.1	1.3	1.4	1.4	1.3	1.7
126	730	532	705	699	666	9.6	20.1	5.9	5.0	10.2	2.2
127	678	672	679	633	666	1.8	.9	2.0	5.0	2.4	0
128	560	555	501	542	540	3.7	2.8	7.2	.4	3.5	1.8
129	488	582	555	519	536	9.0	8.6	3.5	3.2	6.1	8.1
130	578	575	572	577	576	.3	.2	.7	.2	.4	0
131	561	527	524	561	543	3.3	2.9	3.5	3.3	3.2	0
132	583	564	518	560	556	4.9	1.4	6.8	.7	3.4	0
133	484	370	445	463	440	10.0	15.9	1.1	5.2	8.0	12.0
134	501	420	533	565	505	.8	16.8	5.5	11.9	8.8	6.4
135	551	516	498	484	512	7.6	.8	2.7	5.5	4.2	0
136	521	507	536	546	528	1.3	4.0	1.5	3.4	2.6	1.0
137	314	419	483	489	426	26.3	1.6	13.4	14.8	14.0	8.0
138	549	484	445	526	501	9.6	3.4	11.2	5.0	7.3	1.6
139	537	633	536	521	557	3.6	13.6	3.8	6.5	6.9	0
140	523	534	557	562	544	3.8	1.8	2.4	3.3	2.8	2.9
141	554	564	544	525	547	1.3	3.1	.5	4.0	2.2	0
142	560	518	561	573	553	1.3	6.3	1.4	3.6	3.2	.2
143	563	527	550	488	532	5.8	.9	3.4	8.3	4.6	2.0
144	521	555	577	520	543	4.1	2.2	6.3	4.2	4.2	2.6
145	581	510	523	525	535	8.6	4.7	2.2	1.9	4.4	.3
146	519	587	599	568	568	8.6	3.3	5.5	0	4.4	5.0
147	587	570	549	609	579	1.4	1.6	5.2	5.2	3.4	2.9
148	632	616	626	611	621	1.8	.8	.8	1.7	1.3	.4
149	356	458	498	445	439	18.9	4.3	13.4	1.4	9.5	38.2
150	568	560	552	593	568	0	1.4	2.8	4.4	2.2	1.4
151	544	567	546	566	556	2.2	2.0	1.8	1.8	2.0	1.6
152	538	550	538	539	541	.6	1.6	.6	.4	.8	2.4
153	578	520	511	440	512	12.9	1.6	.2	14.1	7.2	13.0
154	505	569	529	473	519	2.7	9.6	1.9	8.9	5.8	4.2
155	485	528	425	438	469	3.4	12.6	9.4	6.6	8.0	24.2
156	444	482	456	476	464	4.3	3.9	1.7	2.6	3.1	9.6
157	453	488	421		454	.2	7.5	7.3		5.0	9.9
158	538	552	501	569	540	.4	2.2	7.2	5.4	3.8	11.3
159	519	457	448	482	476	9.0	4.0	5.9	1.3	5.0	4.4
160	559	550	547	589	561	.4	2.0	2.5	5.0	2.5	2.5
161	612	601	630	577	605	1.2	.7	4.1	4.6	2.6	1.3
162	620	644	619	681	641	3.3	.5	3.4	6.2	3.4	0
163	564	524	502	591	545	3.5	3.9	7.9	8.4	5.9	0
164	664	589	580	623	614	8.1	4.1	5.5	1.5	4.8	.3
165	613	586	527	569	574	6.8	2.1	8.2	.9	4.5	0
166	647	621	599	626	623	3.9	.3	3.9	.5	2.2	.4
167	635	620	569	612	609	4.2	1.8	6.6	.5	3.3	.5
168	577	571	492	592	558	3.4	2.3	11.8	6.1	5.9	2.8
169	618	649	612	612	623	.8	4.2	1.8	1.8	2.2	0
170	624	585	408	617	558	11.8	4.8	26.9	10.6	13.5	13.7
171	556	609	503	551	555	.2	9.7	9.4	.7	5.0	3.4
172	589	563	533	551	559	5.4	.7	4.7	1.4	3.0	0
173	602	526	566	573	567	6.2	7.2	.2	1.1	3.7	0
174	566	503	562	582	553	2.4	9.0	1.6	5.2	4.6	0
175	611	520	396	568	524	16.6	.8	24.4	8.4	12.6	9.0
176	641	534	532	607	578	10.9	7.6	8.0	5.0	7.9	0
177	588	553	451	466	514	14.4	7.6	12.3	9.3	10.9	2.8
178	549	586	519	505	540	1.7	8.5	3.9	6.5	5.2	4.0
179	626	613	666	657	640	2.2	4.3	4.1	2.7	3.3	.3
180	571	563	518	556	552	3.4	2.0	6.2	.7	3.1	13.5
181	601	600	619	612	608	1.2	1.3	1.8	.7	1.2	0
182	630	585	591	592	600	5.0	2.5	1.5	1.3	2.6	0
183	594	505	574	584	564	5.3	10.5	1.8	3.5	5.3	1.8
184	546		513	535	531	2.8		3.4	.8	2.3	17.8
185	539	588	598	571	574	6.1	2.4	4.2	.5	3.3	2.3
186	627	525	597	586	584	7.4	10.1	2.2	.3	5.0	2.9
187	543	457	581	436	504	7.7	9.3	15.3	13.5	11.4	14.2
188	650	561	550	561	580	12.0	3.3	5.2	3.3	6.0	2.0
189	607	507	547	578	560	8.4	9.5	2.3	3.2	5.8	5.2
190	502	388	518	397	451	11.3	14.0	14.9	12.0	13.0	18.2
191	531	520	573	556	545	2.6	4.6	5.1	2.0	3.6	1.6
192	503	393	501	529	482	4.4	18.5	3.9	9.8	9.2	7.4
193	530	465	572	475	510	3.9	8.8	12.2	6.9	8.0	10.2
194	767	674	761	661	716	7.1	5.9	6.3	7.7	6.8	2.1
195	799	696	749	699	736	8.6	5.4	1.8	5.0	5.2	.2
196	793	728	716	753	748	6.0	2.7	4.3	.7	3.4	0
197	816	729	779	741	766	6.5	4.8	1.7	3.3	4.1	0
198	720	680	705	704	702	2.6	3.1	.4	.3	1.6	.6
199	745	727	737	763	743	.3	2.2	.8	2.7	1.5	0
200	735	606	717	603	665	10.5	8.9	7.8	9.3	9.1	.8
201	781	772	753	806	778	.4	.8	3.2	3.6	2.0	0
202	715	581	708	702	676	5.8	14.1	4.7	3.8	7.1	3.9
203	686	741	739	799	741	7.4	0	.3	7.8	3.9	.4
204	691	593	727	728	685	.9	13.4	6.1	6.3	6.7	1.5
205	614	630	740	750	684	10.2	7.9	8.2	9.6	9.0	3.3
206	697	689	655	691	683	2.0	.9	4.1	1.2	2.0	.4
207	682	674	788	779	731	6.7	7.8	7.8	6.6	7.2	2.8
208	700	629	708	772	702	.3	10.4	.9	10.0	5.4	3.0
209	729	671	692	816	727	.3	7.7	4.8	12.2	6.2	0
210	728	691	658	777	714	2.0	3.2	7.8	8.8	5.4	0
211	613	638	759	670	670	8.5	4.8	13.3	0	6.6	7.8
212	662	671	674	735	686	3.5	2.2	1.7	7.1	3.6	0
213	723	575	777	791	716	1.0	19.7	8.5	10.5	9.9	1.8
214	690	633	780	759	716	3.6	11.6	8.9	6.0	7.5	2.6
215	766	722	688	766	736	4.1	1.9	6.5	4.1	4.2	0
216	742	629	717	745	708	4.8	11.2	1.3	5.2	5.6	2.2
217	662	493	759	636	638	3.8	22.7	19.0	.3	11.4	9.6
218	738	686	665	693	696	6.0	1.4	4.5	.4	3.1	0
219	702	687	730	745	716	2.0	4.0	2.0	4.1	3.0	0
220	710	655	690	757	703	1.0	6.8	1.8	7.7	4.3	.5
221	740	721	742	735	734	.8	1.8	1.1	.1	1.0	0
222	713	624	704	762	701	1.7	11.0	.4	8.7	5.4	3.0
223	627	507	729	742	651	3.7	22.1	11.6	14.0	12.8	10.2
224	653	682	694	734	691	5.5	1.3	.4	6.2	3.4	0
225	700	761	747	736	736	4.9	3.4	1.5	0	2.4	0
226	777	665	737	743	730	6.4	8.9	1.0	1.8	4.5	1.8
227	705	807	724	774	752	6.3	7.3	3.7	2.9	5.0	0
228	728	673	697	665	691	5.4	2.6	.9	3.8	3.2	0
229	760	748	604	690	700	8.6	6.9	13.7	1.4	7.6	3.6
230	710	550	710	677	662	7.3	16.9	7.3	2.3	8.4	4.7
231	727	623	798	743	723	.6	13.8	10.4	2.8	6.9	6.2
232	751	721	773	704	737	1.9	2.2	4.9	4.5	3.4	0
233	699	520	752	777	687	1.7	24.3	9.5	13.1	12.2	8.8
234	718	687	712	688	701	2.4	2.0	1.6	1.9	2.0	2.2
235	719	565	655	662	650	10.6	13.1	.8	1.8	6.6	.1
236	797	789	445	770	700	13.9	12.7	36.4	10.0	18.2	7.9
237	628	651	717	663	665	5.6	2.1	7.8	.3	4.0	1.1
238	808	789	758	731	772	4.7	2.2	1.8	5.3	3.5	0
239	730	800	629	747	726	.6	10.2	13.4	2.9	6.8	1.9
240	787	753	727	777	761	3.4	1.1	4.5	2.1	2.8	0
241	727	678	694	662	690	5.4	1.7	.6	4.1	3.0	0
242	753	776	670	758	739	1.9	5.0	9.3	2.6	4.7	.4
243	690	716	663	737	702	1.7	2.0	5.6	5.0	3.6	.8
244	721	705	678	726	708	1.8	.4	4.2	2.5	2.2	0
245	718	717	637	691	691	3.9	3.8	7.8	0	3.9	1.4
246	731	755	683	735	726	.7	4.0	5.9	1.2	3.0	0
247	615	605	596	603	605	1.7	0	1.5	.3	.9	7.4
248	784	713	635	771	726	8.0	1.8	12.5	6.2	7.1	1.6
249	647	666	634	753	675	4.1	1.3	6.0	11.6	5.8	4.2
250	776	603	712	806	724	7.2	16.7	1.7	11.3	9.2	0
251	658	688	447	650	611	7.7	12.6	26.8	6.4	13.4	5.8
252	728	742	570	760	700	4.0	6.0	18.6	8.6	9.3	1.8
253	756	675	630	744	701	7.8	3.7	10.1	6.1	6.9	.4
254	690	606	650	652	650	6.2	6.8	0	.3	3.3	1.4
255	672	708	670	667	679	1.0	4.3	1.3	1.8	2.1	.6
256	595	620	532	664	603	1.3	2.8	11.8	10.1	6.5	2.8
257	542	678	665	684	642	15.6	5.6	3.6	6.5	7.8	5.5
258	645	493	587	642	592	9.0	16.7	.8	8.4	8.7	2.0
259	709	661	658	644	668	6.1	1.0	1.5	3.6	3.0	.2
260	664	651	566	659	635	4.6	2.5	10.9	3.8	5.4	2.8
261	662	588	617	675	636	4.1	7.5	3.0	6.1	5.2	.6
262	682	603	614	699	650	4.9	7.2	5.5	7.5	6.3	1.8
263	520	923	535	688	592	12.2	5.2	9.6	16.2	10.8	3.2
264	673	653	638	647	653	3.1	0	2.3	.9	1.6	.3
265	666	523	515	600	576	15.6	9.2	10.6	4.2	9.9	4.4

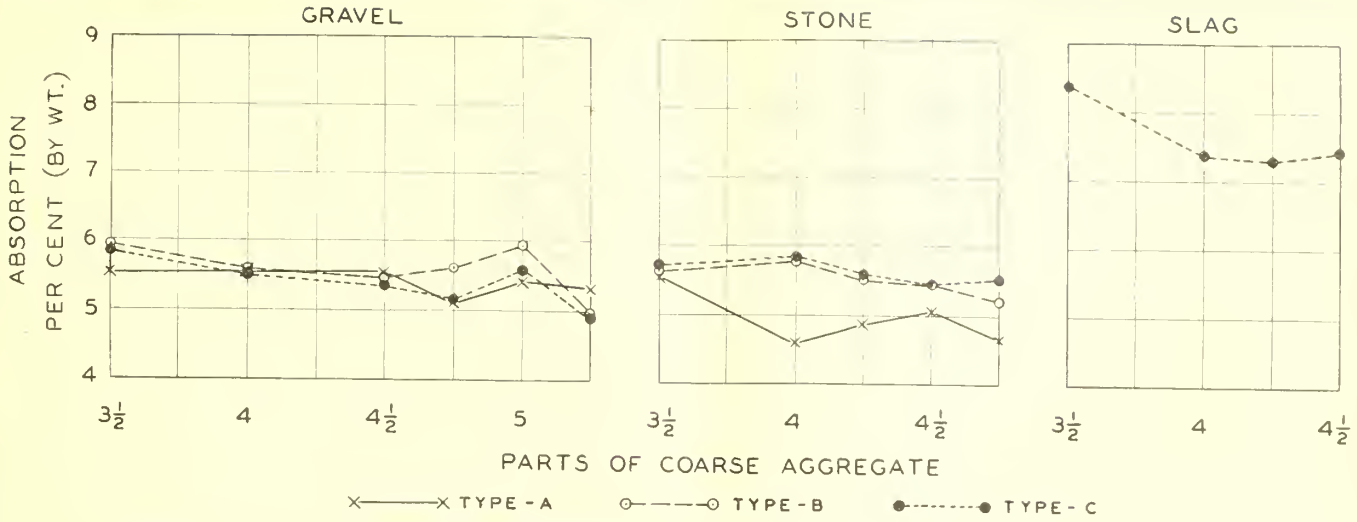


FIGURE 32.—RELATION BETWEEN ABSORPTION OF CORES AT NINE MONTHS AND TYPE OF FINISHING MACHINE

RESULTS OF ABSORPTION TESTS DISCUSSED

In order to develop information which might be of assistance in studying the relative durability of the various mixes, a series of absorption tests was made on 6-inch cores drilled from the test sections. These tests were made in accordance with the standard method calling for 5-hour immersion in boiling water, as specified by the American Society for Testing Materials. The average percentage by weight of water absorbed for each proportion and for each type of finishing machine is given in Figure 32. The average absorption is about 5.5 per cent for all conditions with the exception of the slag concrete, which averages about 7.5 per cent, and the stone concrete finished with type A finisher, which shows about 5 per cent. There is a slight tendency for the absorption to decrease with increasing quantities of coarse aggregate.

These tests are being supplemented now with a series of freezing and thawing tests on a similar series of cores. It is hoped that it will be possible to present a report covering this portion of the investigation at an early date.

STUDIES OF INDIVIDUAL TEST SECTIONS

In order to make it possible to study the manner in which the several variables which were being investigated affected the relative strength of individual sections, it was considered advisable to plot the results of tests section by section. This has been done in Figures 33 to 37, inclusive, in which the results of the various strength tests on each section have been indicated, together with the mix, slump, water-cement ratio, and averge percentages of honeycomb. In each group of three sections the concrete having the lowest slump appears to the left and the concrete having the highest slump to the right. In each group of three sections lines have been drawn between the plotted values of compressive strength and modulus of rupture to indicate the trend of variation of strength with increasing slump. These curves should not be looked upon as indicating a quantative relation, since the abscissa does not represent a variable. The various trends to which attention has been called may easily be traced throughout the entire series by reference to these figures. There are several points to which attention is especially called.

1. For each group of tests a very distinct relation exists between the consistency of the concrete as measured by the slump test and the water-cement ratio. Almost invariably it will be found that, in each group of three sections, a direct relation exists which indicates that the slump test is a very good measure of the relative amount of water in a given concrete, even within the rather narrow limits covered by this study.

2. In general, the additional water required to maintain workability in the mixtures containing high percentages of coarse aggregate does not result in an increase in slump. The upper, left-hand panel of Figure 33 is an exception to this rule.

3. The crushing strength of the concrete in the individual sections follows, in general, the water-cement ratio law. Exceptions may be observed in certain instances, many of which may be explained by the use of concrete which is too dry to be workable. It must be remembered, of course, that in these charts each point represents the average of only two tests in the case of the control specimens and the cores and four tests in the case of the slabs. Individual variations from the general rule are therefore not surprising.

4. There is a marked tendency for the flexural strength of both beams and slabs to fall off for the dry mixes. The tendency is more marked in the case of the slabs, probably because of the fact that under the standard method of molding employed for the control beams it was possible to fabricate more uniform specimens with the drier mixes than by the methods employed in placing the slabs. This tendency is illustrated by reference to sections 3, 17, 25, 32, 50, 71, 85, 122, 149, 180, 211, 230, and 247.

5. In 60 out of a total of 89 groups of three sections each, the section having the highest water-cement ratio showed a slab strength as high as or higher than the corresponding section in the same group having the lowest water-cement ratio. Attention is called to the fact that in each group of three sections the range in water-cement ratio was only sufficient to produce a variation in consistency from dry to medium and that in no case was the water-cement ratio high enough to produce concrete of wet or sloppy consistency.

It should also be noted that whereas the drier mixes would for the most part be rated as workable by laboratory standards, such mixes were not in general found to be workable under the methods of placing and finishing pavement slabs now commonly used. The dry mixes may therefore be considered as lying outside

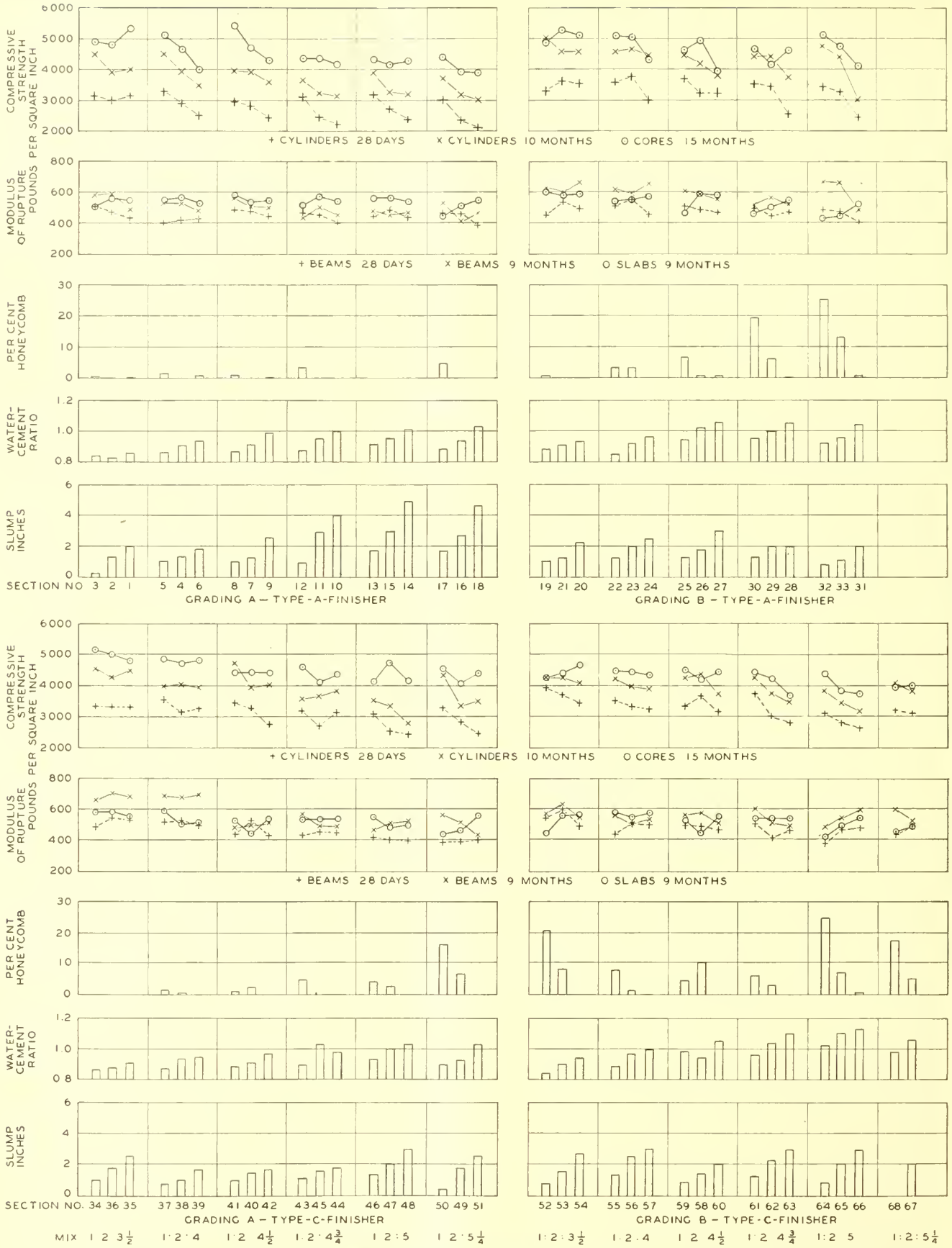

COMPRESSIVE STRENGTH POUNDS PER SQUARE INCH
6000
5000
4000
3000
2000
+ CYLINDERS 28 DAYS
x CYLINDERS 10 MONTHS
O CORES 15 MONTHS
MODULUS OF RUPTURE POUNDS PER SQUARE INCH
800
600
400
200
+ BEAMS 28 DAYS
x BEAMS 9 MONTHS
O SLABS 9 MONTHS
PER CENT HONEYCOMB
30
20
10
0
WATER-CEMENT RATIO
1.2
1.0
0.8
SLUMP INCHES
6
4
2
0
SECTION NO 3 2 1 5 4 6 8 7 9 12 11 10 13 15 14 17 16 18
GRADING A — TYPE-A-FINISHER
19 21 20 22 23 24 25 26 27 30 29 28 32 33 31
GRADING B — TYPE-A-FINISHER
SECTION NO. 34 36 35 37 38 39 41 40 42 43 45 44 46 47 48 50 49 51
GRADING A — TYPE-C-FINISHER
52 53 54 55 56 57 59 58 60 61 62 63 64 65 66 68 67
GRADING B — TYPE-C-FINISHER
MIX 1 2 3½ 1:2:4 1:2 4½ 1:2:4¾ 1:2:5 1:2:5¼
1:2:3½ 1:2.4 1 2 4½ 1:2 4¾ 1:2 5 1:2:5¼

FIGURE 33.—COMPARISON OF VALUES OF COMPRESSIVE STRENGTH, MODULUS OF RUPTURE, PER CENT HONEYCOMB, WATER-CEMENT RATIO, AND SLUMP FOR ALL SECTIONS OF GRAVEL CONCRETE, ROUND 1

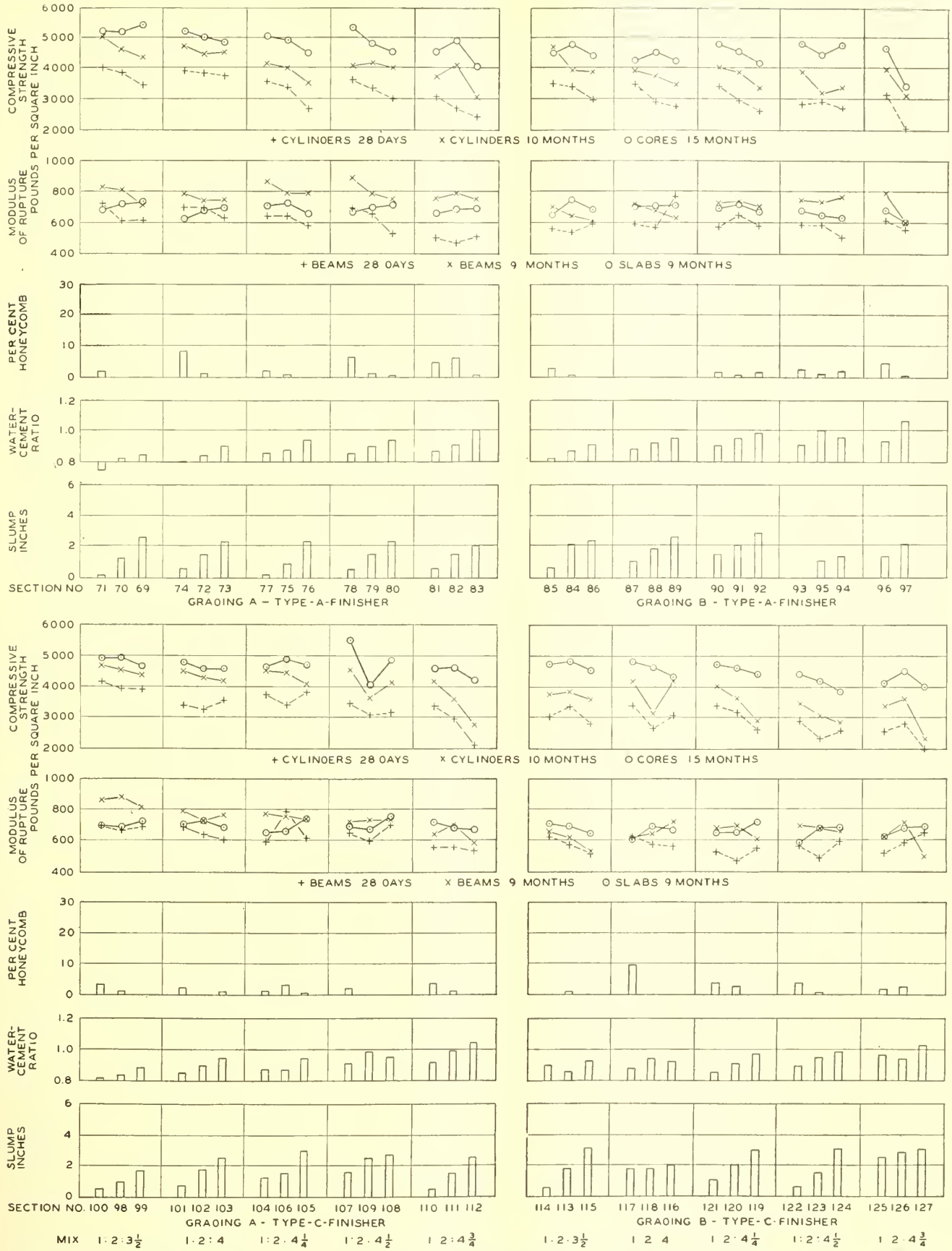


FIGURE 34.—COMPARISON OF VALUES OF COMPRESSIVE STRENGTH, MODULUS OF RUPTURE, PER CENT HONEYCOMB, WATER-CEMENT RATIO, AND SLUMP FOR ALL SECTIONS OF STONE CONCRETE, ROUND 1

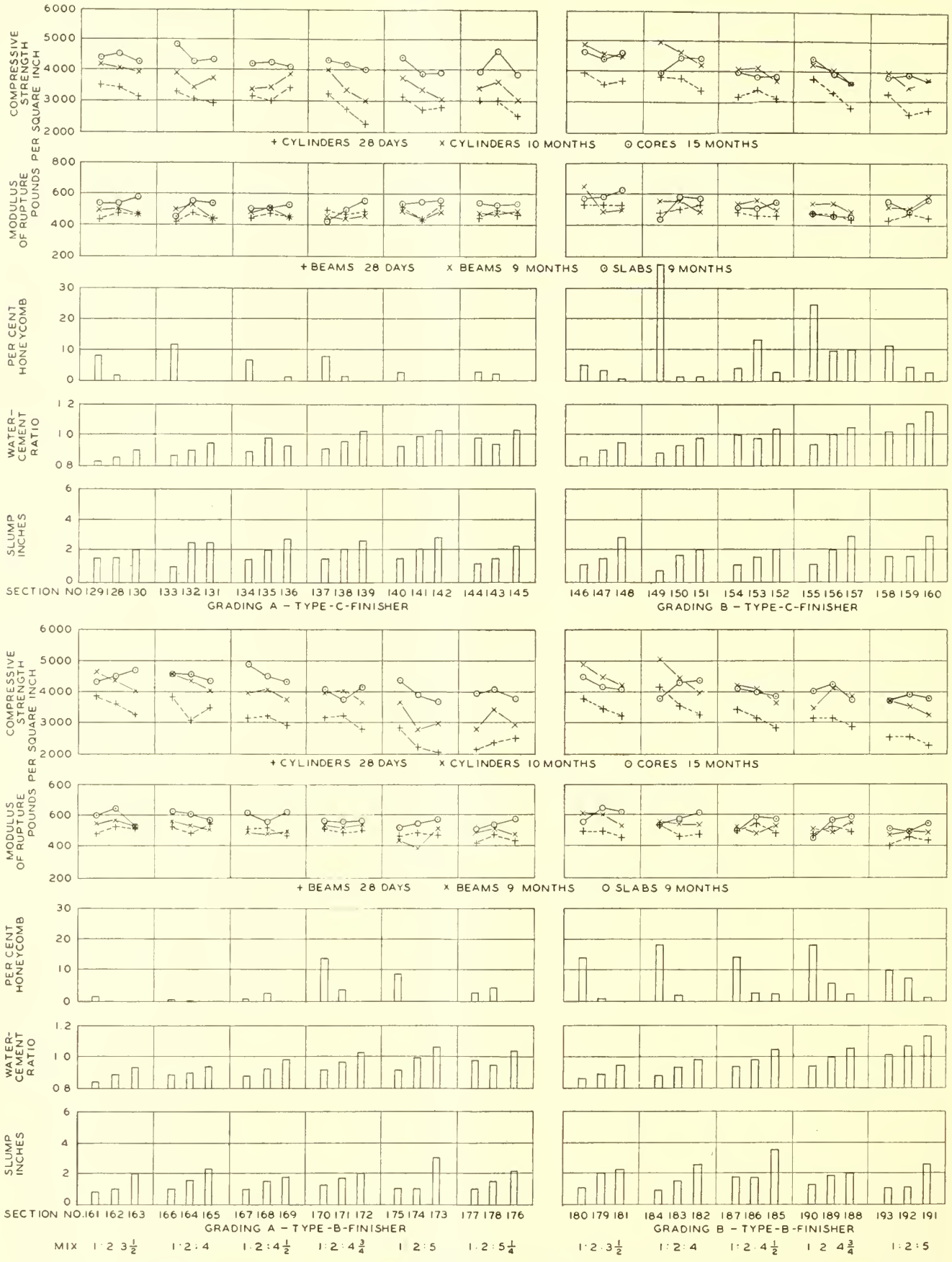


FIGURE 35.—COMPARISON OF VALUES OF COMPRESSIVE STRENGTH, MODULUS OF RUPTURE, PER CENT HONEYCOMB, WATER-CEMENT RATIO, AND SLUMP FOR ALL SECTIONS OF GRAVEL CONCRETE, ROUND 2

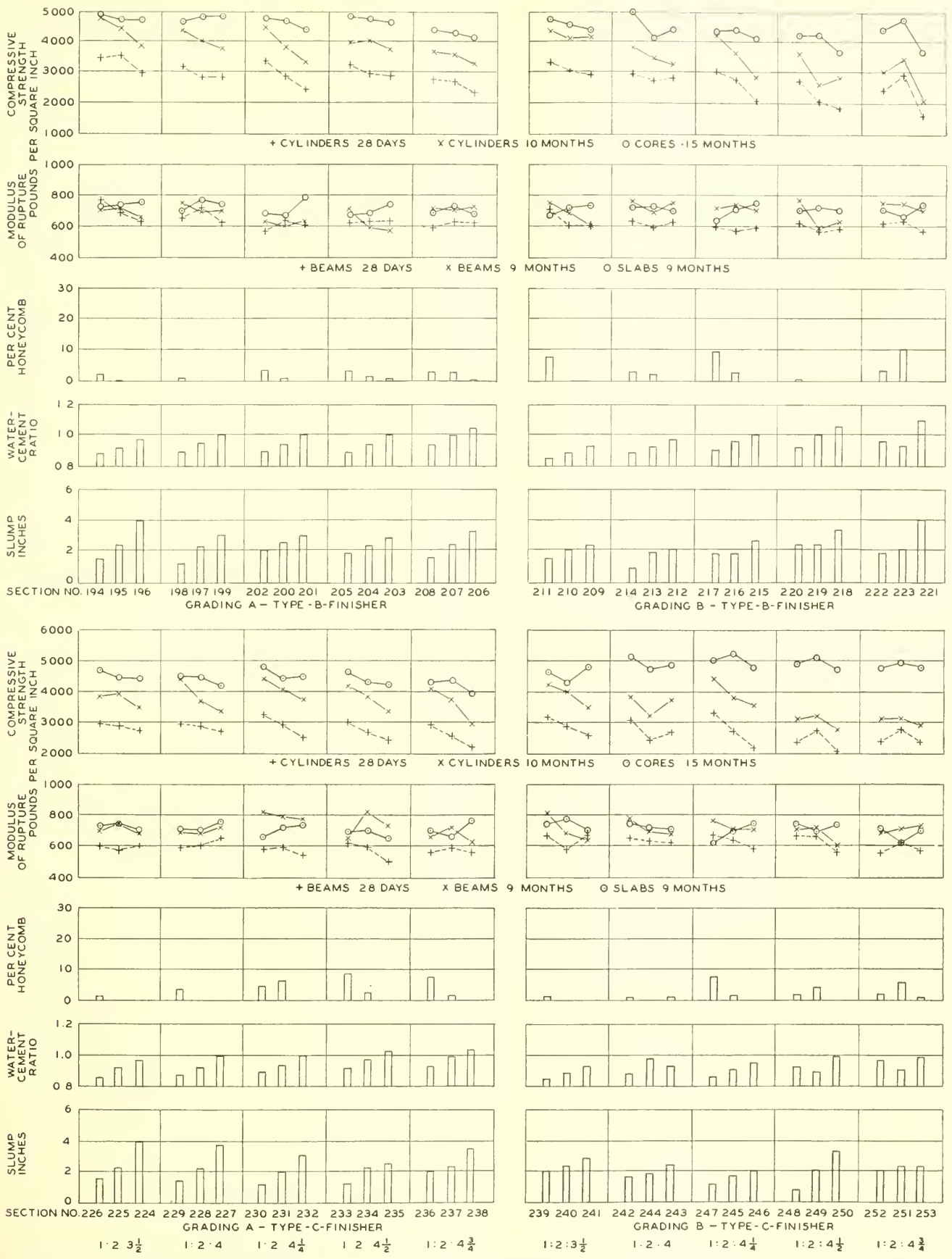

COMPRESSIVE STRENGTH POUNDS PER SQUARE INCH
+ CYLINDERS 28 DAYS
X CYLINDERS 10 MONTHS
O CORES 15 MONTHS
MODULUS OF RUPTURE POUNDS PER SQUARE INCH
+ BEAMS 28 DAYS
X BEAMS 9 MONTHS
O SLABS 9 MONTHS
PER CENT HONEYCOMB
WATER-CEMENT RATIO
SLUMP INCHES
SECTION NO. 194 195 196 198 197 199 202 200 201 205 204 203 208 207 206
GRADING A – TYPE-B-FINISHER
211 210 209 214 213 212 217 216 215 220 219 218 222 223 221
GRADING B – TYPE-B-FINISHER
COMPRESSIVE STRENGTH POUNDS PER SQUARE INCH
+ CYLINDERS 28 DAYS
X CYLINDERS 10 MONTHS
O CORES 15 MONTHS
MODULUS OF RUPTURE POUNDS PER SQUARE INCH
+ BEAMS 28 DAYS
X BEAMS 9 MONTHS
O SLABS 9 MONTHS
PER CENT HONEYCOMB
WATER-CEMENT RATIO
SLUMP INCHES
SECTION NO. 226 225 224 229 228 227 230 231 232 233 234 235 236 237 238
GRADING A – TYPE-C-FINISHER
1:2 3½ 1:2:4 1:2 4¼ 1 2 4½ 1:2:4¾
239 240 241 242 244 243 247 245 246 248 249 250 252 251 253
GRADING B – TYPE-C-FINISHER
1:2:3½ 1:2:4 1:2:4¼ 1:2:4½ 1:2:4¾

FIGURE 36.—COMPARISON OF VALUES OF COMPRESSIVE STRENGTH, MODULUS OF RUPTURE, PER CENT HONEYCOMB, WATER-CEMENT RATIO, AND SLUMP FOR ALL SECTIONS OF STONE CONCRETE, ROUND 2

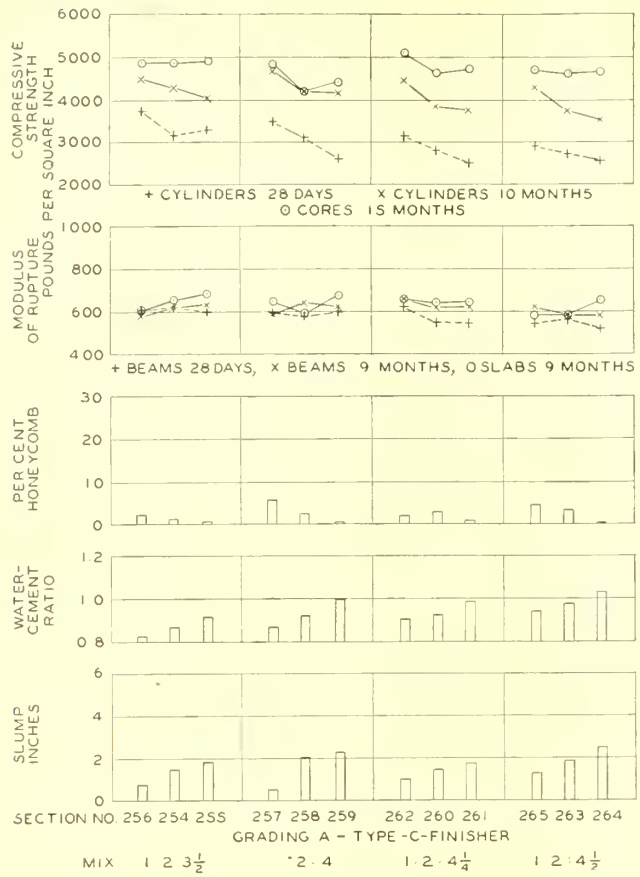


FIGURE 37.—COMPARISON OF VALUES OF COMPRESSIVE STRENGTH, MODULUS OF RUPTURE, PER CENT HONEYCOMB, WAR-CEMENT RATIO, AND SLUMP FOR ALL SECTIONS OF SLAG CONCRETE

of the range of plastic mixes to which the water-cement ratio law applies, and this fact accounts for the falling off in strength.

6. In these same 60 groups, the section in each group having the lowest water-cement ratio was honeycombed in all cases. On the other hand, in 40 out of the 60 groups, honeycomb was entirely absent from the section having the highest water-cement ratio, with 13 out of the remaining 20 showing less than 1 per cent honeycomb.

7. The slump for the driest mix in each group averaged from one-fourth inch to 1 inch, while the slump for the corresponding section in each group having the highest water-cement ratio averaged from 2 to 3 inches.

The above indications point emphatically to the danger of using mixes of less than 2-inch slump in concrete paving work with the methods of finishing in common use to-day. The use of medium consistencies such as were obtained in these tests with a 2 to 3 inch slump will not only give as high or higher slab strength than the drier mixes but will greatly decrease the tendency to honeycomb and thus will promote uniformity.

8. In general, the presence of honeycomb in the slabs, although very seriously affecting the flexural strengths of the slabs themselves, was not accompanied by lower crushing strengths of the cores drilled from the slabs. Neither was there any relation between the extent of honeycomb in the test slabs and the amount of honeycomb in the cores drilled therefrom. These observations lead to the conclusion that neither the crushing strength nor freedom from honeycomb of cores is necessarily an indication of the homogeneity of the concrete in the slab.

CONCLUSIONS STATED

For the materials, proportions, and methods of placing used in these tests the results of the investigation justify the following conclusions:

1. For a constant sand-cement ratio an increase in the quantity of coarse aggregate beyond the limits ordinarily employed in practice decreased the strength of the concrete in the pavement substantially in proportion to the amount of additional water required to maintain workability.

2. For corresponding increases in the water-cement ratio the percentage of reduction in flexural strength was somewhat less than the percentage of reduction in crushing strength.

3. The workability and uniformity in strength of the concrete was decreased in proportion to the amount of coarse aggregate added.

4. The workability of the concrete was greatly reduced by decreasing the maximum size and at the same time increasing the amount of fines in the gravel coarse aggregate, even though the corresponding variations in the percentage of voids in the gravel and in the value of $\frac{b}{b_0}$ for the concrete were small.

5. For the gravel concrete the reduction in workability for any mix caused by the use of the poorly graded gravel was greater than the reduction in workability caused by increasing the percentage of coarse aggregate in the concrete containing the well graded material. (See fig. 22.)

6. For a given consistency as measured by the slump test, different gradations of gravel produced different degrees of workability. (Fig. 23.)

7. For a given gradation of coarse aggregate the consistency as determined by the slump test measured the workability of the concrete.

8. For a given proportion, higher slab strengths were obtained with concrete having a medium consistency (2 to 3 inches slump) than with very dry concrete (¼ to 1 inch slump).

9. In order to obtain satisfactory uniformity and freedom from honeycomb it was necessary to use a consistency corresponding to a slump of approximately 2 to 3 inches.

10. The various types of finishing machine employed in these tests gave approximately the same results.

11. For concrete reasonably free from honeycomb, the strength of the control beams appeared to be a satisfactory measure of the strength of the corresponding pavement slab.

12. Neither the amount of honeycombing observed in the drilled cores or the crushing strengths developed by the drilled cores measured the extent of honeycomb in the pavement slabs or the flexural strength developed by the pavement slabs.

RECOMMENDATIONS

On the basis of the information produced by these tests, the following recommendations are made:

a. That all specifications for concrete for pavements contain a definite requirement covering consistency. The slump test is recommended for this purpose.

b. That the use of paving mixes showing less than 2-inch slump should be discouraged.

c. That specifications for consistency of paving concrete to be finished by any of the methods now in use be revised when necessary to provide that the concrete have a consistency corresponding to a slump of from 2 to 3 inches.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.
Report of the Chief of the Bureau of Public Roads, 1929.
Report of the Chief of the Bureau of Public Roads, 1930.

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TECHNICAL BULLETIN

No. 55T. Highway Bridge Surveys.

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MISCELLANEOUS CIRCULARS

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*93M. Direct Production Costs of Broken Stone. 25c.

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MISCELLANEOUS PUBLICATION

No. 76MP. The Results of Physical Tests of Road-Building Rock.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio. (1927)

Report of a Survey of Transportation on the State Highways of Vermont. (1927)

Report of a Survey of Transportation on the State Highways of New Hampshire. (1927)

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio. (1928)

Report of a Survey of Transportation on the State Highways of Pennsylvania. (1928)

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D– 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.

Vol. 5, No. 19, D– 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.

Vol. 5, No. 24, D– 6. A New Penetration Needle for Use in Testing Bituminous Materials.

Vol. 11, No. 10, D–15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID AND EMERGENCY ROAD CONSTRUCTION

AS OF

JULY 31, 1931

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION						BALANCE OF FEDERAL AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		ESTIMATED TOTAL COST	FEDERAL AID ALLOTTED	EMERGENCY ADVANCE FUND	MILEAGE			ESTIMATED TOTAL COST	FEDERAL AID ALLOTTED	EMERGENCY ADVANCE FUND	MILEAGE				
					INITIAL	STAGE*	TOTAL				INITIAL	STAGE*	TOTAL		
ALABAMA	2,130.9	$ 8,955,589.62	$ 3,303,349.82	$ 1,533,095.32	229.9	106.7	336.6	$ 83,312.53	$ 48,656.28	$ 34,691.58	1.4	2.7	4.1	$ 3,272,118.88	ALABAMA
ARIZONA	881.2	6,932,100.56	3,887,127.50	983,500.88	260.0	189.8	449.9	84,143.72	54,813.77	13,904.20	2.5	5.1	7.8	389,825.20	ARIZONA
ARKANSAS	1,751.1	8,813,129.59	4,183,963.44	1,318,157.00	230.6	102.0	332.5	1,433,760.98	684,625.65		30.4	29.9	60.3	46,942.04	ARKANSAS
CALIFORNIA	1,835.3	12,122,600.78	5,085,074.10	2,733,179.23	315.8	108.7	426.5	2,589,848.52	978,511.28	218,622.21	75.1	1.5	76.6	337,393.88	CALIFORNIA
COLORADO	1,343.1	5,913,200.75	3,189,288.16	1,357,384.65	213.8	87.6	311.4	420,677.29	224,411.60	73,000.00	13.5	12.9	26.4	1,788,436.71	COLORADO
CONNECTICUT	280.0	4,981,188.80	1,855,739.03	520,491.00	51.1		51.1	85,753.26	43,361.82		.2		.2	4,388.65	CONNECTICUT
DELAWARE	306.0	1,041,247.28	518,889.13	400,000.00	52.6		52.8	180,024.00	90,012.00		7.4		7.4	73,518.71	DELAWARE
FLORIDA	539.9	8,527,676.69	3,058,613.83	1,042,604.35	188.8		188.8							1,611,780.08	FLORIDA
GEORGIA	2,795.5	9,638,502.58	4,564,215.68	1,844,815.35	328.5	120.3	448.8	3,327,238.55	1,380,655.00	400.00	41.3	82.3	123.6	72,047.46	GEORGIA
IDAHO	1,299.1	4,092,380.38	2,304,237.71	953,187.90	273.2	41.0	314.2	588,694.92	344,782.83	30,000.00	35.6	44.6	80.1	338,802.77	IDAHO
ILLINOIS	2,341.3	26,886,068.88	12,319,599.80	2,985,175.48	806.6	31.8	838.5	5,492,122.77	2,431,405.76	384,100.00	183.2	4.7	187.9	111,317.87	ILLINOIS
INDIANA	1,679.0	12,990,286.18	5,890,234.06	1,890,269.33	354.1		354.1	1,526,586.29	763,290.61	152,081.48	74.3		74.3	81,364.18	INDIANA
IOWA	3,142.6	7,707,081.74	3,261,723.87	2,090,547.88	167.2	82.5	249.7	228,965.73	74,810.23			7.0	7.0	4,058.10	IOWA
KANSAS	3,219.3	7,962,557.83	3,818,244.74	1,818,143.58	478.8	46.5	525.4	2,088,202.46	883,635.94	80,509.31	98.3	36.8	135.1	8,190.70	KANSAS
KENTUCKY	1,867.0	6,643,969.18	2,882,004.78	1,268,690.50	268.2	124.8	393.1	570,813.13	273,043.45	150,100.00	42.4	15.0	57.4	148,529.37	KENTUCKY
LOUISIANA	1,460.8	7,725,357.51	3,658,561.75	1,147,827.00	208.2	19.9	228.1	751,852.47	265,033.41			.4	.4		LOUISIANA
MAINE	610.4	4,463,890.05	1,709,858.83	868,464.50	101.5		101.5	1,426,923.17	597,157.58	20,000.00	24.8		24.8	332,495.47	MAINE
MARYLAND	707.0	1,830,463.15	898,580.88	561,112.37	60.1	4.2	64.3	484,348.30	155,381.33		15.6		18.8		MARYLAND
MASSACHUSETTS	726.2	10,141,004.40	2,583,922.51	1,011,460.00	89.4		89.4	2,245,296.43	824,811.12	130,000.00	23.8		23.8	884,683.37	MASSACHUSETTS
MICHIGAN	1,786.8	12,427,342.26	5,270,855.21	1,907,462.99	366.6	51.2	417.7	1,440,888.88	621,981.60	98,000.00	66.5	1.8	68.3	1,481,915.00	MICHIGAN
MINNESOTA	3,971.0	8,005,316.98	3,511,057.31	2,159,993.00	74.8	302.8	377.7	436,553.78	200,740.40	90,000.00	5.1	14.8	19.9	37,280.53	MINNESOTA
MISSISSIPPI	1,765.3	4,262,333.39	2,100,381.31	1,429,736.00	184.6	87.8	272.4	275,748.23	136,432.87	6,000.00	.1	12.8	12.7	3,737,808.93	MISSISSIPPI
MISSOURI	2,674.2	9,711,425.99	3,886,078.09	2,375,333.55	262.3	73.5	335.8	980,001.25	420,530.03	69,371.38	25.8	10.7	36.5	114,404.25	MISSOURI
MONTANA	1,838.0	10,889,446.08	5,134,075.28	1,568,403.11	806.8	143.5	950.4	1,698,725.81	967,250.58	7,600.00	76.8	72.0	148.8	1,381,452.64	MONTANA
NEBRASKA	3,857.8	9,984,022.50	4,675,696.90	1,573,839.72	269.3	210.4	479.7	1,854,078.65	892,110.26	111,852.90	87.2	58.5	146.7	358,854.60	NEBRASKA
NEVADA	1,105.4	2,862,836.57	1,853,366.18	862,286.13	91.1	253.4	344.5	528,851.49	417,947.53	135,567.49	17.5	31.0	48.5	135,605.64	NEVADA
NEW HAMPSHIRE	387.8	1,702,782.58	608,709.41	365,000.00	20.5	5.1	38.8	62,697.76	28,181.83	55,000.00	1.4	2.1	3.6	102,460.80	NEW HAMPSHIRE
NEW JERSEY	586.4	6,438,011.40	1,647,370.30	1,033,572.44	83.8		83.8	1,064,842.63	520,149.17	3,038.41	6.9	.6	7.5	631,954.64	NEW JERSEY
NEW MEXICO	1,992.7	5,831,630.10	3,529,782.93	1,096,589.90	233.4	88.8	332.3	107,277.63	67,700.35		.2		.2	118,929.92	NEW MEXICO
NEW YORK	2,701.9	39,114,210.33	14,146,258.00	3,914,566.00	734.8	7.0	741.8	4,741,800.00	1,701,495.22	54,000.00	84.0		84.0		NEW YORK
NORTH CAROLINA	1,869.5	8,555,687.66	3,182,285.97	1,736,601.68	204.0	64.6	268.5	596,181.75	265,079.74	58,000.00	35.1	4.0	38.1	1,599,497.42	NORTH CAROLINA
NORTH DAKOTA	4,598.2	3,732,298.80	1,910,368.15	891,314.85	411.9	315.7	727.6	983,015.78	497,783.41	74,109.00	65.3	238.5	303.8	872,833.87	NORTH DAKOTA
OHIO	2,582.1	15,443,910.72	5,120,835.94	2,525,015.96	261.3	22.7	284.0	4,360,393.12	1,637,221.93	473,522.04	68.0	32.9	100.2	677,607.33	OHIO
OKLAHOMA	2,011.3	8,047,780.09	3,851,227.47	1,775,760.06	297.7	121.1	418.8	457,401.02	237,297.68	90,537.36	6.8	10.8	16.8	72,517.96	OKLAHOMA
OREGON	1,285.8	7,574,826.53	4,211,931.47	1,149,997.89	265.7	82.8	348.5	419,000.40	216,273.88	104,414.51	31.5	8.4	39.9	367,606.50	OREGON
PENNSYLVANIA	2,575.1	13,154,512.65	6,668,408.88	3,234,280.51	301.0		301.0	2,140,567.22	976,307.55	278,882.49	70.8		70.8	685,822.19	PENNSYLVANIA
RHODE ISLAND	220.4	2,427,028.37	907,421.64	334,000.00	37.2		37.2							118,133.37	RHODE ISLAND
SOUTH CAROLINA	1,869.8	5,028,123.08	2,286,629.98	1,042,586.00	78.2	136.5	214.7							255,042.63	SOUTH CAROLINA
SOUTH DAKOTA	3,854.3	5,347,157.85	2,814,090.88	1,188,363.14	326.0	184.3	510.3	632,942.43	341,183.23	15,000.00	.3	107.8	108.1	103,536.96	SOUTH DAKOTA
TENNESSEE	1,531.4	3,220,348.87	1,577,146.83	1,237,000.21	131.5	9.5	141.0	380,641.70	190,320.84		28.0		28.0	1,842,657.37	TENNESSEE
TEXAS	7,041.3	18,903,882.64	8,160,075.15	4,796,835.33	786.3	323.0	1,109.3	3,592,649.33	1,559,903.88	185,000.00	137.0	64.8	191.8	2,532,268.93	TEXAS
UTAH	1,041.0	2,101,308.31	1,182,748.80	634,674.35	113.7	65.8	179.6	519,359.34	343,155.24	72,262.45	24.8	40.8	65.8	503,683.78	UTAH
VERMONT	301.8	1,498,025.70	608,121.37	400,000.00	37.1	4.7	41.8							3,138.16	VERMONT
VIRGINIA	1,647.2	5,730,133.96	2,583,323.11	1,338,362.06	241.0	46.4	287.4	565,847.70	282,923.85		21.2		21.2	602,645.31	VIRGINIA
WASHINGTON	988.2	5,838,510.96	2,608,229.56	1,187,441.66	176.4	36.3	211.7	518,090.02	256,863.39	15,000.00	14.1	7.7	21.8	827,152.35	WASHINGTON
WEST VIRGINIA	760.4	6,694,614.72	2,296,189.88	847,554.00	136.5	25.4	161.8	366,373.41	158,782.94	27,830.00	8.0		9.0	176,305.72	WEST VIRGINIA
WISCONSIN	2,438.6	8,260,061.49	3,562,416.37	1,860,910.00	223.5	70.8	294.1	1,547,527.77	452,166.90	96,301.21	60.5	15.3	75.8		WISCONSIN
WYOMING	1,657.0	4,583,090.07	2,725,277.21	940,157.83	343.0	222.7	565.7	235,693.88	117,946.00	78,000.00	17.2	32.2	49.4	171,498.31	WYOMING
HAWAII	47.5	1,429,014.40	580,743.37	394,528.83	41.1		41.1	285,652.62	97,414.67		3.8		3.8	1,506,429.60	HAWAII
TOTALS	89,870.8	389,003,532.09	173,362,384.15	72,036,333.69	12,207.3	4,034.0	16,241.3	64,645,245.07	23,850,221.68	3,464,136.99	1,628.4	1,004.0	2,632.4	30,543,555.80	TOTALS

(*) THE TERM STAGE CONSTRUCTION REFERS TO ADDITIONAL WORK DONE ON PROJECTS PREVIOUSLY IMPROVED WITH FEDERAL AID. IN GENERAL, SUCH ADDITIONAL WORK CONSISTS OF THE CONSTRUCTION OF A SURFACE OF HIGHER TYPE THAN WAS PROVIDED IN THE INITIAL IMPROVEMENT.

U. S. GOVERNMENT PRINTING OFFICE 1931

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 12, NO. 7 ▽ SEPTEMBER, 1931

APPROACH TO HOLLAND TUNNEL, JERSEY CITY

COURTESY OF NEW JERSEY STATE HIGHWAY COMMISSION

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

VOL. 12, NO. 7 SEPTEMBER, 1931 G. P. St. CLAIR, Editor

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THE SOIL PROFILE AND THE SUBGRADE SURVEY

Reported by W. I. WATKINS, Assistant Soil Surveyor, U. S. Bureau of Chemistry and Soils, and HENRY AARON, Assistant Highway Engineer, U. S. Bureau of Public Roads

SUBGRADE soil profiles, the importance of which with respect to road construction is discussed in this report, are studied and mapped in the field according to procedures suggested by the pedologist. However, the highway engineer is interested only in those soil properties upon which the service ability of pavements depend. Some of these properties are often of minor significance in an agricultural sense. In other cases the scale adopted in the publication of reports of agricultural soil surveys is too small to show all of them. For this reason attention in field and laboratory is directed specifically to those properties which are important in road construction. The final map of the subgrade soil profile discloses those engineering properties of soils which are determined by laboratory tests, as well as those physical characteristics which are identified by the soil scientist.

Information furnished by the soil scientist which assists the engineer both in the study of subgrade profiles and in the interpretation of existing soil data is contained in the reports listed in the attached bibliography.[1]

THE SOIL PROFILE DESCRIBED

According to Dr. C. F. Marbut (*1*) the soils of the United States may be divided into two large categories: (1) Pedalfers and (2) pedocals. The pedalfers are soils that tend to accumulate iron and aluminum and have no horizon[2] of lime carbonate accumulation, even if the soils have limestone as parent material. The accumulation of lime carbonate and other salts is a characteristic feature of pedocals. This discussion is based on studies covering a wide range of pedalfer and a few pedocal soil types established by the Bureau of Chemistry and Soils.

A vertical section of the weathered soil layers, having different characteristics though uniformly weathered within themselves, is known as the soil profile. In soil technology the term "soil profile" applies to the weathered layers which constitute what is known as the solum and the material immediately underlying it. In the well-drained upland soils of the humid region the solum is ordinarily less than 10 feet in thickness and is usually composed of a lighter-textured upper and a heavier-textured lower layer, both of which may have sublayers.

A soil type is the soil unit and is as definite an object as any other classified object, animal, or plant, and always presents approximately the same physical characteristics (*5*, *6*). A soil series is a group of soil types that have profiles uniform in all respects except the texture of the surface layer.

Practically all modern soil research is conducted upon the basis of the soil-type profile. There is no desire to create the impression that every soil type or soil layer differs widely from all other soil types or soil layers; but it is believed that the method of classification based on soil types furnishes the most economical basis for studying soils and utilizing the information obtained for any given purpose. After the properties or behavior of the layers of a soil type are once determined the data obtained will be immediately and continuously available and may be of use in connection with studying any branch of science in any way pertaining to soil, such as agriculture, erosion, corrosion, drainage, road construction, and so on. Such information is applicable wherever the soil type from which the information was obtained is found. The behavior of soil layers common to the individual soil types of a soil series or of a closely related soil series may thus be ascertained by determining the behavior of such a common layer in one or more of the closely related soil types. Additional discussion of the soil profile is contained in a report by Joffe (*2*). Figures 1 to 4, inclusive, illustrate the character of the material encountered in the determination of soil profile.

SOIL PROFILE AS RELATED TO ROAD CONSTRUCTION

One can readily understand that the characteristics of the soil profile control percolation, capillarity, seepage—in fact, all internal water movement except that influenced by nonsoil agencies, such as animal burrows, root holes, etc. The influence of the soil layers on the nature of water movement will change if the soil is in any way disturbed. The degree of change will depend on the extent to which the natural characteristics are destroyed. The resistance to change of the natural characteristics and the subsequent effect on soil behavior are governed largely by the relative amounts of silt and colloids, and the chemical composition of the colloids (*7*). Certain other constituents such as diatoms and mica flakes have a decided influence upon the physical behavior of a soil (*8*). The soil characteristics, their descriptive terms, and the influence they may have on subgrade soil behavior are summarized in the following paragraphs:

Texture.—For sand or sandy loams: Coarse, medium, fine, very fine. Other soils classified by texture as loam, silt loam, clay loam, silty clay loam, clay. May reflect plasticity, density, slumping, porosity, expansion, contraction, capillarity, susceptibility to compaction, elasticity, percolation, water absorption, erosion, structure.

Color.—Black, red, brown, yellow, drab, gray, mottled. May reflect organic content, oxidation, leaching, chemical content, structure, erosive qualities.

Structure.—Buckshot, granular, columnar, cloddy, crumb, adobe, dense, massive, laminated, nut, mealy, structureless. May reflect capillarity, percolation, porosity, water absorption, internal drainage, bulking on being disturbed, susceptibility to compaction, elasticity.

Consistency.—Brittle, hard, compact, tough, tenacious, sticky, plastic, cheesy, friable, mellow, loose. May reflect capillarity, percolation, porosity, water absorption, internal drainage, bulking, compactibility, elasticity, supporting power.

Compactness.—Clay pan, hardpan; soils also classified by degree of compaction. May give idea of bulking, internal water movement, degree of support.

[1] Italic figures in parentheses refer to the reports listed in the bibliography.

[2] In standard soil descriptions for humid regions the light-textured surface layer is designated the A horizon, the underlying heavier-textured layer as the B horizon, and the third layer, consisting of the unweathered or incompletely weathered geological formation, as the C horizon. The A and B horizons constitute the real soil profile, the Solum horizon of Frosterus, while the C horizon is part of the parent geological formation not made significantly lighter or heavier in texture by soil-making processes. An horizon may have subhorizons such as A_1, A_2, etc. For highway purposes, the various layers are numbered consecutively from the surface down without designating any horizon. For example, instead of layers A_1, A_2, B, etc., we have layers 1, 2, 3, etc. Some soil surveyors designate these layers as zone 1, zone 2, zone 3, etc.

Cementation.—Classified by cementing material and degree of cementation. May give idea of bulking, internal water movement, degree of support.

Chemical composition.—Significant chemicals are iron, calcium, concretions, alkali, sesquioxides, exchangeable bases, pH value. May affect structure, adhesiveness, friability, plasticity, drainage and water movement, formation of hardpans, expansion, contraction, water retention, heat of wetting.

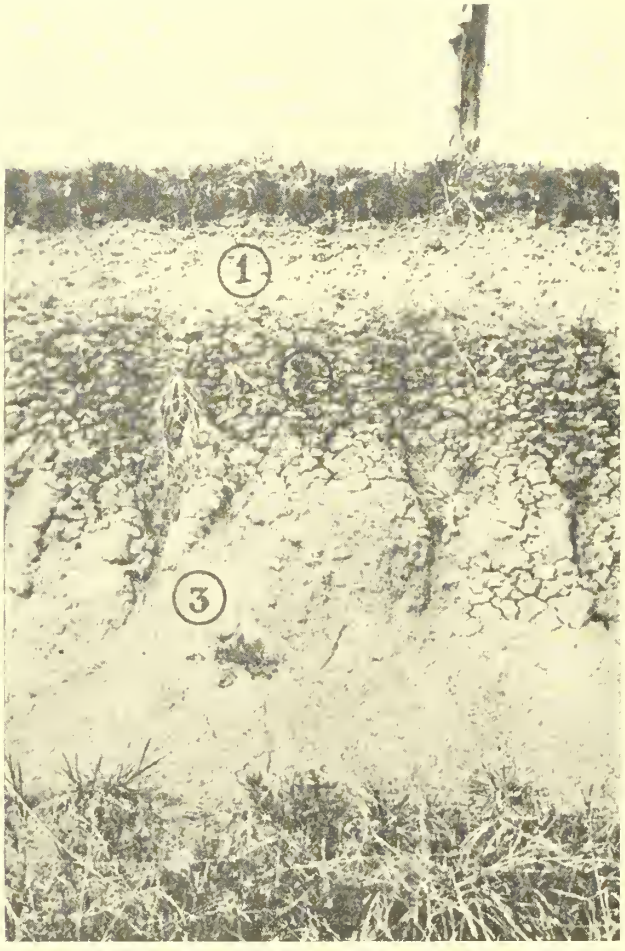

FIGURE 1.—PROFILE OF IREDELL SILT LOAM SHOWING (1) LIGHTER-TEXTURED UPPER SOIL LAYER, (2) HEAVY, PLASTIC, STICKY, WAXY CLAY, AND (3) DECOMPOSED ROCK

Organic content classified by kind of material and degree of decomposition; influences water absorption, temperature, firmness, stability, compactibility and springiness.

In nonalkali soils the percentage of colloids and their chemical composition are probably the most important factors controlling such soil characteristics as consistency, structure, and porosity. This is especially true of the silica sesquioxide ratio. Soils containing colloids with a low silica sesquioxide ratio, i. e., colloids high in the sesquioxides of iron and aluminum and low in the oxides of silica, shrink and expand but slightly at extremes of moisture content, are not very plastic or excessively sticky, and are more friable, more porous, and less plastic than soils containing an equal amount of colloids with a high silica sesquioxide ratio (*9, 10*). Some highly colloidal clays of low sesquioxide ratio make better road subgrades than other soils containing less colloids with a high silica sesquioxide ratio (*11*).

Texture has usually been taken as an index of the rate of percolation, but structure is now considered of equal or greater importance (*12*), and the destruction of the natural structure may increase or decrease the rate of percolation. The variability in percolation rates of the various soil layers or soil material causes and controls seepage or lateral water movement. Seepage develops when the amount of water percolating through any one soil layer exceeds the percolating rate of that below. The amount of lateral movement is governed by the amount of water and differences in rate of perco-

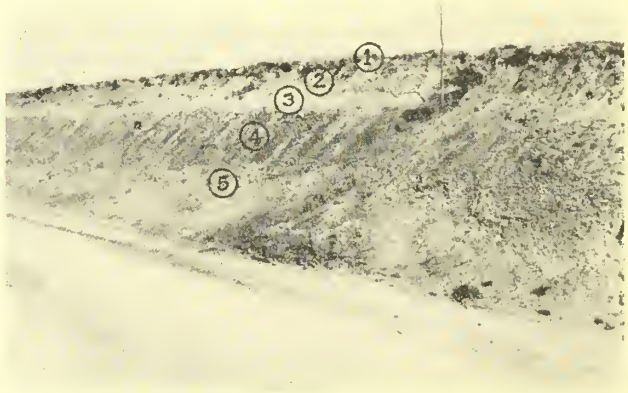

FIGURE 2.—PROFILE OF LEONARDTOWN SILT LOAM SHOWING (1) GRAYISH BROWN GRANULAR SILT LOAM, (2) BROWN, GRANULAR SILTY CLAY LOAM, (3) GRAY, COMPACT, SANDY HARDPAN, (4) THE FRIABLE CLAY, AND (5) THE COMPACT CLAY

lation. It will be seen that water movement depends not only on the character of any particular layer but also on its relation to other associated layers and the effect of local climatic conditions. A silt loam may block or retard percolation in a soil profile where the silt is overlain by sands or may act as a water carrier if underlain by compacted sands or clays. In some soil profiles clay layers may act as water carriers if underlain by denser, less pervious layers. Stratified soils, dense clays, and hardpans are frequently causes of seepage. Silts, sands, and gravels are the most prolific carriers. Cleavage

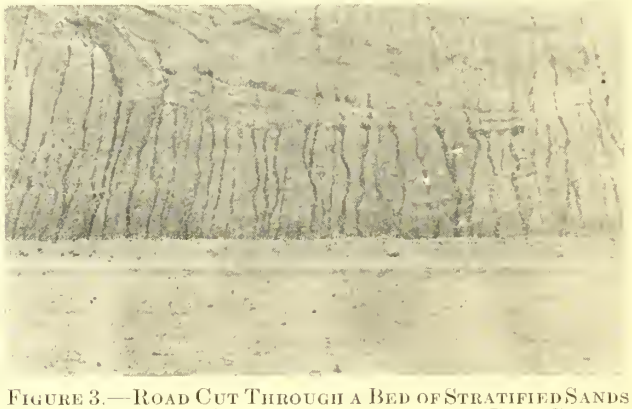

FIGURE 3.—ROAD CUT THROUGH A BED OF STRATIFIED SANDS AND GRAVELS IN GLACIAL DRIFT. NOTICE HOW GRAVEL DEPOSIT ENDS ABRUPTLY AT RIGHT OF PHOTOGRAPH

planes in geological materials are also channels for lateral water movement. Figure 5 shows cleavage planes in shale. The soil profile discloses these water-carrying layers in the field. Through its control of water movement, the soil profile furnishes a basis for the design of drains. The proper location of drains with respect to the soil profile is illustrated in Figure 6.

The study of soil profiles has shown frost boils to be confined to certain soil materials or soil layers by themselves, or in combination with other soil materials or

FIGURE 4.—A CUT THROUGH COLLINGTON LOAM. THE MATERIAL OUTLINED IN THE LOWER PART OF THE PHOTOGRAPH CONTAINS DIATOMS, IS POROUS AND LIGHT IN WEIGHT IN ITS NATURAL CONDITION, HOLDS ENORMOUS AMOUNTS OF WATER, HAS LITTLE STABILITY WHEN WET AND IS DISTINGUISHED BY A SOAPY FEEL

layers possessing certain definite characteristics. These studies have shown the very fine sands, silt loams, and silty clay loams having little or no apparent soil structure to be the greatest sources of frost boils where these soils or soil materials are found associated with water carriers or where they act as water carriers themselves. In New Hampshire, for example, frost boils were found to be associated mainly with stratified silty clay loams and very fine sandy loams that retarded percolation or had seepage characteristics, and carried the water into the subgrade, where it froze.

In Minnesota frost boils are found to occur most frequently in soil material having no readily determinable structure, and vary in degree of severity depending on the soil material and source of water. The more variable the soil and the more water available, the more severe the frost boil development. The extremely variable glaciated soil materials of this region probably develop the most severe heaving. Less severe heaving occurs in the structureless loessial soil material of southeastern Minnesota. Figure 7 is a diagrammatic illustration of a frost boil and its relation to the soil profile of a glaciated soil, while Figure 8 shows the same for a loessial soil. These drawings represent actual cases.

This structureless loessial soil material is a silt loam containing a high percentage of very fine sand, has a high water-holding capacity, is unstable when wet, and has a texture likely to produce maximum active capillarity. The material is underlain by a heavy, slightly pervious clay that restricts percolation and causes a water table to develop just above the clay. This water table may supply the water to the road surface by capillarity.

Capillarity is controlled by structure and density as well as by texture. The soil types overlying the loessial soil material seldom develop frost boils if in their natural condition, but if disturbed and used as fill in poorly drained areas are likely to become soft during the spring, as silty soils do not have resistant structure and are likely to have more active capillarity when the structure is destroyed.

FIGURE 5.—STRATIFICATIONS IN SHALE. THE CONTACT PLANES BETWEEN THE DIFFERENT SHALE STRATA ARE POTENTIAL PLANES OF WATER SEEPAGE

The soil profile offers a means for the study of the design and construction of fills and back slopes in different soil types and under different drainage conditions. With a knowledge of the physical characteristics of the various layers of the soil profile, the stability of the different layers in place or when combined in fill may be determined. This information will establish the slope of fills and back slopes which will

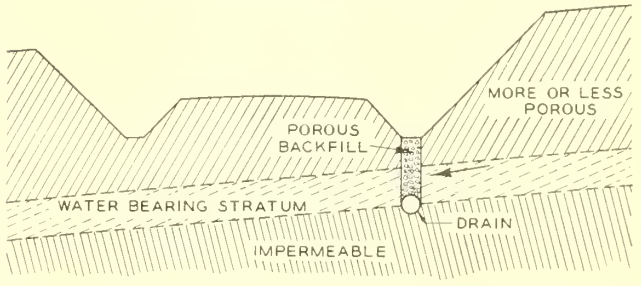

FIGURE 6.—DRAINS SHOULD EXTEND THROUGH THE ENTIRE DEPTH OF THE PERVIOUS LAYER AND INTO THE IMPERVIOUS LAYER

have to conform to the layer or material having the smallest angle of repose; or remedies will have to be adopted to increase its angle of repose. There is a possibility of establishing fairer grading contracts through a knowledge of the amount of the various soils or soil materials which are to be moved, as such data make possible a closer estimate of the cost of handling.

The characteristics of a soil and its value as a subgrade under different conditions are directly reflected in the condition of the road surface. Rigid pavements are affected by inequalities of subgrade support. Nonrigid pavements are chiefly affected by low road supporting power. Studies have shown that the subgrade

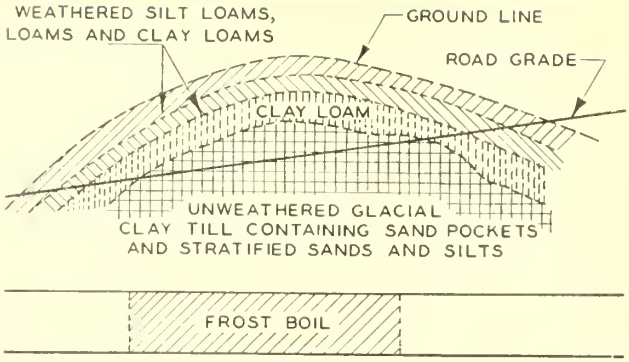
WEATHERED SILT LOAMS, LOAMS AND CLAY LOAMS
GROUND LINE
ROAD GRADE
CLAY LOAM
UNWEATHERED GLACIAL CLAY TILL CONTAINING SAND POCKETS AND STRATIFIED SANDS AND SILTS
FROST BOIL

FIGURE 7.—FROST BOIL IN GLACIATED SOIL MATERIALS

soil exerts important influence upon the distance between transverse cracks in concrete roads, and that excessive longitudinal cracking develops on definite layers of certain soil types. Other soil characteristics, such as swelling, affinity for water, rebound, etc., are detrimental to concrete before it sets as it is then a flexible pavement. Figures 9 and 10 show the relation which exists between the various soil zones of a soil type and the pavement condition. In addition to the effect of the soil itself, these results are affected by climate, rainfall and temperature range. With a knowledge of these factors it is possible to control the effects on the pavement by subgrade preparation and slab design. The importance of the subgrade soil properties is especially observed in the performance of oil and gravel road surfaces, since a portion of the soil is incorporated with the surfacing material. By using the soil profile as a basis for correlating the different types of road deterioration or condition, it has been

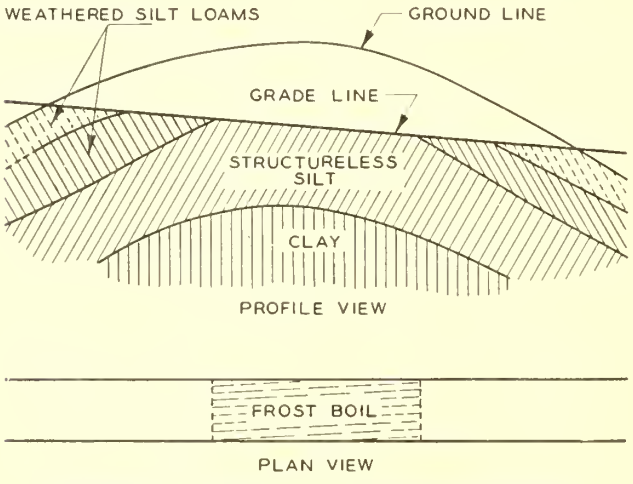

FIGURE 8.—FROST BOIL IN STRUCTURELESS SILT

possible to determine under what conditions and upon what soil layer or soil material of a soil type the different kinds of road failures can be expected.

The correlation of the condition of roads in service with the soil profile and the determination of the physical characteristics of the layers composing the profile as disclosed by laboratory tests form the basis for the grouping of subgrade soils according to performance. The success obtained by these correlation studies shows that the soil profile is a reliable, scientific, and economical means of approaching and solving subgrade soil problems. Furthermore, the soil profile offers a method for the practical application of subgrade research to highway construction.

DETERMINATION OF THE SOIL PROFILE

The soil profile is determined by examining the soil in its natural field condition. This work is best accomplished by examining excavations, road cuts, etc., but the method of using a soil auger is the most common. There is no definite rule to follow in making these examinations, except that the soil should be examined at intervals close enough to determine the soil type and by borings deep enough to penetrate the more or less non-uniform layers of soil or soil material.

According to Doctor Marbut (*4*) the examination of the soil section, after the locality has been determined upon, should proceed in a systematic way somewhat as follows:

1. Texture.[3]—The successive layers or horizons differing in texture, or in fineness or coarseness of the material, should be carefully examined. The examination should extend to a depth of at least 5 or 6 feet. The texture of each layer and its thickness should be described.

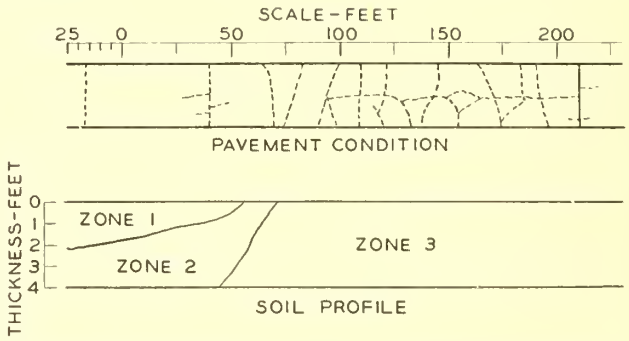

FIGURE 9.—RELATION OF VARIOUS SOIL ZONES TO PAVEMENT CONDITION

2. Color.—The successive layers which can be differentiated according to differences in color, should be noted. Each layer should be described and its thickness given.

3. Structure.—The several layers that differ according to structure should be examined carefully, structure being defined as the kind and size of soil particle aggregation. Special note should be made of horizons with (*a*) fine granular structure (granules about the size of bird shot or smaller); (*b*) coarse granular structure (granules ranging up to half an inch or more in diameter and usually more angular or irregular in shape than the granules making up the fine granular structure); (*c*) layered or platy structure, in which the material splits into thin plates (not to be confused with stratification); (*d*) buckshot structure, in which the soil on drying breaks up into angular fragments (found to characterize heavy clays usually having a considerable percentage of lime); and (*e*) single-grain structure in which the material is like flour or sand with no aggregation of particles.

4. Consistency.—A determination should be made of the successive layers or horizons differing in consistency (stickiness, friability, plasticity). A description of each should be given and its thickness noted.

5. Compactness.—The relative compactness of the several layers should be determined, as measured by the degree of resistance to the penetration of a pointed instrument.

6. Cementation.—It should be determined whether or not resistance to penetration is due in any horizon to

[3] The terms used by the pedologist in describing the various layers of the soil profile are defined in Appendix A of this report.

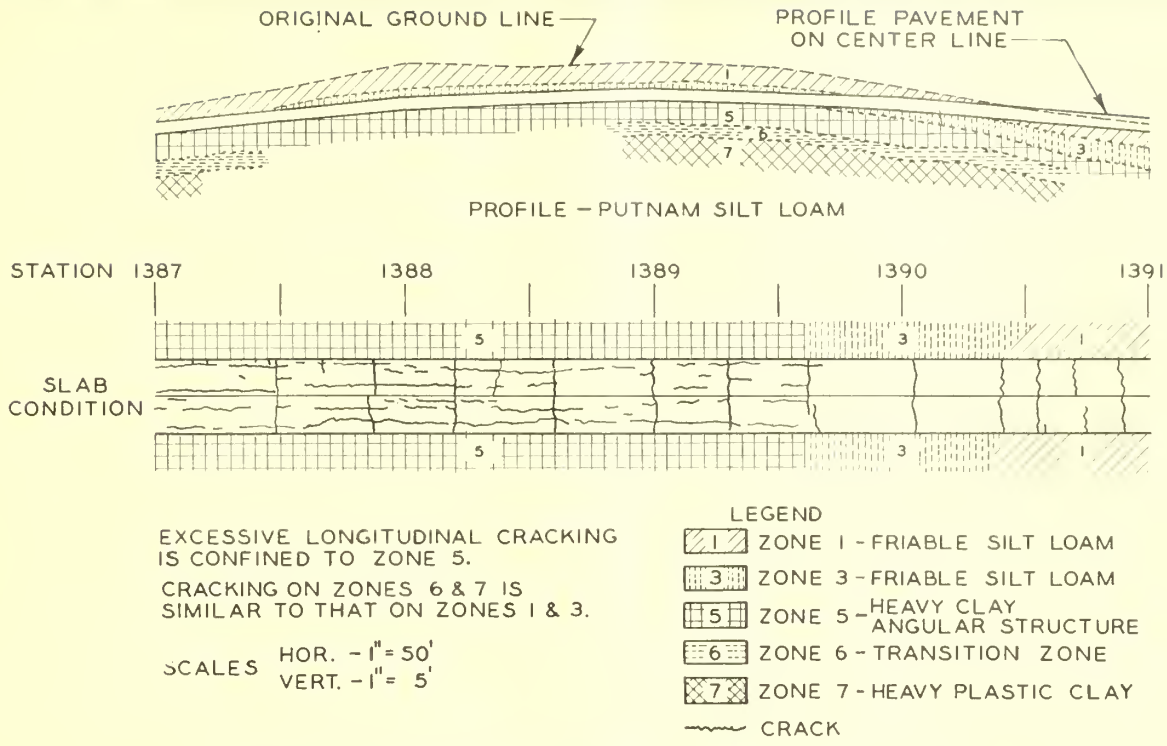
ORIGINAL GROUND LINE
PROFILE PAVEMENT ON CENTER LINE
PROFILE — PUTNAM SILT LOAM
STATION 1387
1388
1389
1390
1391
SLAB CONDITION
EXCESSIVE LONGITUDINAL CRACKING IS CONFINED TO ZONE 5.
CRACKING ON ZONES 6 & 7 IS SIMILAR TO THAT ON ZONES 1 & 3.
SCALES HOR. - 1" = 50'
VERT. - 1" = 5'
LEGEND
ZONE 1 - FRIABLE SILT LOAM
ZONE 3 - FRIABLE SILT LOAM
ZONE 5 - HEAVY CLAY ANGULAR STRUCTURE
ZONE 6 - TRANSITION ZONE
ZONE 7 - HEAVY PLASTIC CLAY
CRACK

FIGURE 10.—CRACKING WHICH DEVELOPED DURING CURING PERIOD

cementation. If so, the probable cementing material (light colored or reddish, very strongly cemented or weakly cemented) should be ascertained.

7. Chemical composition.—While the determination of the chemical composition of the various parts of the soil section or profile can not be performed in the field by the usual field methods, there are certain features that may be determined in at least a qualitative way. Field examination can detect the presence of horizons with concentrations of organic matter or of salts of the alkalies and alkaline earths.

The organic matter referred to here is that contained in the soil and not that lying on the soil. Of this there are two kinds to be looked for. (*a*) The organic matter in the surface soil is recognized by the dark color, and the approximate relative percentage present is indicated by the intensity of the dark color. The determination of the thickness of the dark-colored layer in the virgin soil is very important. (*b*) In some soils, usually confined to regions with a cool, moist climate, there is present, at a depth ranging from 6 inches or less to somewhat more than a foot, a layer of brown or coffee-brown organic matter forming a film 6 or 8 inches in thickness.

The salts of the alkalies and alkaline earths accumulate in the soil under favorable conditions. Since the work here contemplated is general and the soil characteristics dealt with are those of wide regional distribution, we may practically neglect all salts except the carbonate of lime. The more soluble salts constituting what is usually known as "alkali" are present in relatively small areas and may be neglected, or the soils in which they occur may be designated merely as alkali soils.

Horizons of lime carbonate accumulation may be identified readily by anyone and should be looked for where the rainfall is less than 17 to 18 inches per year in cool to cold climates and 30 inches per year in hot or very warm climates. The unweathered material beneath the soil in any region, arid, subhumid, or humid may have a high percentage of lime carbonate, but such material should not be confused with the horizon of true lime carbonate accumulation.

Sesquioxides accumulate in the soil under favorable conditions. Since accumulations of aluminum hydroxide are not readily identified by the usual field methods these may be left out of consideration. We are concerned, therefore, with accumulations of iron oxides.

Iron oxides occur in two forms: The first form consists of accumulations of finely divided or colloidal iron oxide (hydroxide). The degree of concentration may be determined, within a rather wide range of error, by the intensity of the red color. The existence of red horizons in the soil profile should be noted and should be illustrated with samples, even though they be small. The second type consists of accumulations of iron oxide concretions or large masses, usually porous or slaglike. This statement does not refer to ironstone slabs or ferruginous sandstone layers which may be found in many places in the parent geological formations. The accumulations referred to here are to be found either in the B horizon or at the top of the C horizon. In hot countries they take the form of thick masses of porous slaglike iron oxide lying at depths ranging from somewhat less than 3 to more than 15 feet. They may consist of fragments scattered over the surface.

MAPPING THE SOIL PROFILE

The detailed mapping of soil profiles is accomplished in the following manner:

1. Vertical soil sections are examined at frequent intervals and classified into layers according to the method described above (*4*). The interval at which soil examinations are made depends on the uniformity of the soil examined.

2. The limits of the various layers are plotted as shown in Figure 11, A, alternate numbers being used to indicate the layers. At test hole No. 1, the con-

secutive layers are numbered 1, 3, 5, etc., so as to allow the inclusion of any other layer which might enter the profile within the section mapped. At test hole No. 4, layer 2 is mapped between layers 1 and 3. This requires the examination of another test hole (No. 5), or perhaps several, between Nos. 3 and 4, to determine the horizontal limits of layer 2.

3. The profile is completed by connecting the points marking the limits of the layers as shown in Figure 11, B.

DETAILS OF THE SUBGRADE SURVEY DESCRIBED

The purpose of the subgrade survey is to furnish the engineer with significant information on the following subjects:

1. The final location of the road both vertically and horizontally.
2. The selection of suitable fill material.
3. The design of the roadway section.
4. The design and location of ditches, culverts, and drains.
5. The need for subgrade treatment and the type required.
6. The selection of the type of road surface and its design.

The subgrade survey consists of three parts: (*a*) The determination of the soil profile, (*b*) the determination of the physical properties of the soils included in the profile, and (*c*) the mapping of the profile in order to supply information important to road design.

The determination of the physical properties of soils and soil materials and their grouping according to performance has been discussed in previous reports (*8, 13*).

The method of making the subgrade survey depends on the type of information required. Two types of subgrade survey are made, (*a*) surveys to furnish information with respect to roads in service, and (*b*) surveys to furnish information with respect to the design of new roads.

EQUIPMENT

The following equipment is required to make subgrade surveys:

One 3-foot soil auger and three 3-foot extensions as illustrated in Figure 12.
Two small pipe wrenches.
One light pick.
One shovel.
A supply of sample bags.
A supply of tags for marking samples.
A ball of twine.
One engineer's level.
One hand level.
One 12-foot level rod, three-section.
One 100-foot metallic tape.
One 12 by 15 inch strip of stiff cardboard.
One roll of 20-inch cross-section paper, 10 divisions to the inch each way.
Notebooks.
A supply of survey stakes.
One camera and supply of films.
A supply of keel.

SUBGRADE SURVEY TO OBTAIN INFORMATION REGARDING ROADS IN SERVICE

The section for study having been selected, the procedure is carried out in the following manner:

1. The section is staked out, the original construction stations being used if possible. Arbitrary stations will serve the purpose when it is not convenient to locate original stations.

2. Cross-sections are taken every 50 feet along the center line or oftener if topography requires, and for a distance of 150 feet on each side of the center line. Elevations are obtained with an engineer's level to the nearest tenth of a foot. The accuracy of an engineer's level is necessary for the construction of center-line and bank-line profiles, but a hand level is sufficiently accurate for the topography adjacent to the highway. An assumed elevation may be used as a bench mark.

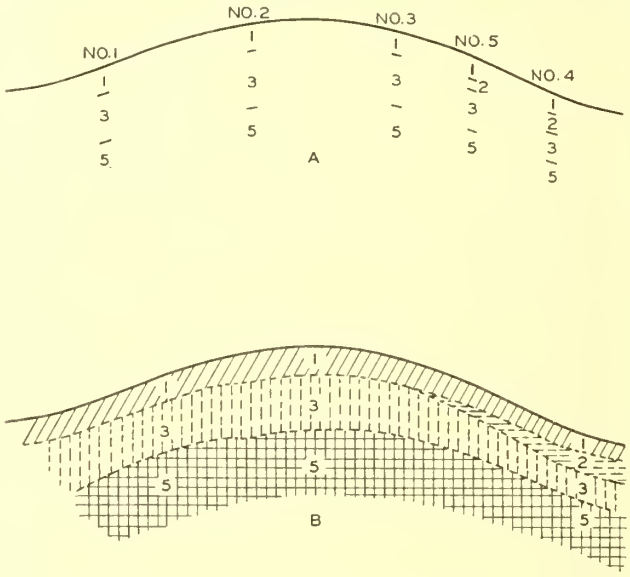

FIGURE 11.—EXAMPLE OF SOIL PROFILE MAPPING, ILLUSTRATING THE DETERMINATION OF LIMITS OF INTERMEDIATE LAYERS BY SUPPLEMENTARY BORINGS

3. A plan of the roadway is drawn, showing the type of pavement, type of failure, portion of roadway which is built over an older road, if any, and any special construction. A scale of 50 feet to the inch is used.

4. Cross-section notes are plotted to the same scale as the plan of roadway and contours are drawn in by interpolation.

5. The bank-line profiles are drawn and the center-line profile is projected upon them, as well as the grade line of the preexisting road, if any. The horizontal scale is 50 feet to the inch. The vertical scale is 5 feet to the inch. Ordinary cross-section paper, 20 inches wide with 10 divisions to the inch each way, is the most convenient type of record sheet. Sheets are cut to a length of 30 inches, folded two ways and clamped to the sheet of stiff cardboard 12 inches wide and 15 inches long.

6. The soils are mapped and the profiles plotted according to the procedure outlined previously. The desired information is obtained and recorded in the following manner:

a. The back slopes are scraped down so that the original undisturbed material is exposed and the limits of the various layers are plotted on the prepared profile sheet. This work is supplemented by soil-auger boring so that a profile is obtained to a depth of at least 3 to 5 feet below the center-line grade. Any variations in moisture content are specially noted. The depth will vary with the uniformity of the soil layers or soil material. The elevations of the limits of the different layers in the exposed back slopes are obtained by means of a hand level, the elevation of the center line being used as a bench mark.

b. In a separate notebook each layer is described in detail according to the nomenclature of the committee on terminology (*3*). (See Appendix A.) The relative imperviousness or porosity is also recorded.

c. Examinations of the soil are made every 50 feet or less, depending upon the uniformity of the profile. (See discussion of Figure 11.)

d. On the plan of the roadway are plotted the limits of the various soil layers found directly under the surfacing material. When the roadway is cut through uniform layers of soil material, the limits are obtained by constructing cross-sections from the bank-line profiles. When the roadway is constructed of fill material or cut through a heterogeneous soil material, the limits are determined by soil-auger borings in the shoulder.

7. A 5-pound sample of soil from each layer is obtained from the exposed back slopes with a pick and shovel, placed in a canvas bag, tied securely, marked with proper identification, and shipped to the laboratory. A sufficient number of samples are taken to determine the range in test results for what appears to be the same layer.

8. Photographs are taken illustrating the condition of the pavement, the shoulders, the back slopes, the ditches, and the appearance of the soil layers.

9. The data collected in the manner described above are analyzed, together with the laboratory test results, and information on the following subjects is developed:

a. The relation that exists between the pavement condition, the field characteristics of the soil, and the physical properties of the soil as determined in the laboratory.

b. The possible reasons for failure.

c. Possible curative measures for the case under examination.

d. Preventive measures which may be applied in the future.

A sample report of a subgrade survey of a road in service is given in Appendix B of this report to illustrate the above procedure.

SUBGRADE SURVEY TO OBTAIN INFORMATION WITH RESPECT TO THE DESIGN OF A NEW ROAD

Before starting a subgrade survey of this kind the engineer should make a study of all the existing information on the soil types in that vicinity. Wherever maps prepared by the Bureau of Soils[4] are available, they should be carefully studied, and the limits of the various soil types and their characteristics should be noted. It must be kept in mind that the detail to which such maps are carried is not particularly adapted to a subgrade survey. Nevertheless they give a clear idea of the variations which will be encountered. Where this information is not available, a reconnaissance survey should be made of the soil materials on existing highways which parallel the new highway, noting the changes in soil as shown in exposed cuts. The notes should include a description of each soil type according to the nomenclature of the committee on terminology. (*3*) The value of the information obtained from this rough survey lies in the fact that similar soil conditions may be expected to accompany similar topographic features.

After this information has been digested the survey proceeds in the following manner:

1. The profile of the ground line and the proposed grade line are constructed on the same type of sheet as was specified for subgrade surveys of existing pavements.

2. Borings are made with a soil auger at frequent intervals and each soil type is classified into layers, as described under the heading "Determination of the Soil Profile" (p.184).

a. Spacing of borings.—The spacing of the borings will vary with the uniformity of the profile and the topography. A convenient interval, such as the even stations, may be assumed at the beginning. This interval may be varied under the following conditions: (1) If the profile is uniform, the interval may be increased. (2) When the character of the profile changes, intermediate borings should be made until it is clear that all variations have been mapped. (3) Where topography is rolling and grade changes rapidly from cut to fill, borings are necessary only in the cuts. (4) Where the original ground line or old road grade is to be covered with fill material, no examination is necessary except to determine the character of the support. If the fill material is to be obtained from borrow ditches along the road, the soil should be examined to the entire depth of the borrow.

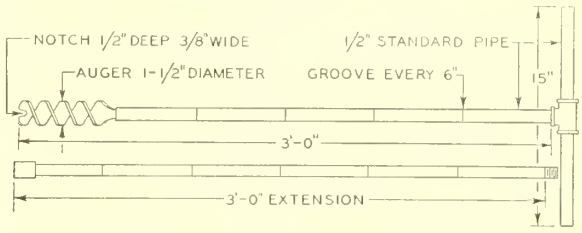

FIGURE 12.—SOIL AUGER AND EXTENSION

b. Depth of borings.—The borings should generally be carried to a depth of at least 3 feet below the grade line. The depth may vary in accordance with the following stipulations: (1) When the road lies within the uniform layers of the soil profile, the boring should extend down to the first layer below the ditch line which would block percolation, or through a pervious layer which would carry water. (2) When fill material is to be borrowed from ditches alongside the road, the boring should extend at least to the estimated depth of borrow. (3) In the study of frost action the borings should extend to the mean depth of frost in those soil materials showing a high affinity for frost accumulation (see p. 183) and in localities where high water tables prevail.

c. When the located line is over an old road, the soils are mapped by examining the exposed cuts. This work is supplemented by borings.

3. A notation is made of the direction of surface drainage with respect to the proposed roadway.

4. The data obtained from the borings is plotted on the prepared profile sheet. On this sheet are indicated the limits of the several types and layers, the relative moisture content at various depths, and the location of culverts and drains.

5. In a separate notebook the field characteristics of each layer are described according to the nomenclature prepared by the committee on terminology. The relative imperviousness or porosity of each layer is indicated.

6. From each layer and type encountered at least a 2-pound sample of soil is taken for laboratory classification. A sufficient number of samples should be obtained to determine the range in test results for what

[4] The former Bureau of Soils is now a part of the Bureau of Chemistry and Soils.

appears to be the same layer. These samples are placed in canvas bags, tied securely, marked with station number and layer, and shipped to the laboratory. The boring of more than one hole may be necessary to obtain the required sample.

7. Recommendations regarding the design of the road surface are made on the basis of the known behavior of pavements for which the conditions of soil, climate and topography are similar.

8. After the road is graded, a final check is made on the soil as exposed by grading operations.

The final form of a subgrade survey sheet submitted to the engineer in charge of design is included in Appendix B of this report.

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APPENDIX A

TERMS IDENTIFYING SOILS IN THE PROFILE

The following terms [1] are among those used in describing the various layers of the soil profile. They definitely identify the field characteristics of the soil material as found in its natural state. In the compilation of these terms the report of the committee on terminology of the American Soil Survey Association was largely used.

[1] The terms describing structure, consistency, compactness, cementation, and chemical composition have not yet been standardized, nor have they been adopted by the division of soil survey, Bureau of Chemistry and Soils. They may be considered tentative and approximate. More accurate definitions are not yet available.

TEXTURE

Texture is a term indicating the size of the individual soil grains or particles and the proportions of material of each size present in any given case.

As the soil is usually made up of particles of widely varying size, the textural terms express the average effect or the combined effect of all these grain sizes. They may indicate the predominance (in quantity or in textural effect) of a certain group of grains.

Texture is determined by mechanical analysis, a laboratory process of separating the soil into groups of grain sizes. The system of mechanical analysis used by the Bureau of Chemistry and Soils separates the soil material into seven grain sizes or "separates" having the following sizes and names:

2 to 1 millimeter, fine gravel.
1 to 0.5 millimeter, coarse sand.
0.5 to 0.25 millimeter, sand.
0.25 to 0.10 millimeter, fine sand.
0.10 to 0.05 millimeter, very fine sand.
0.05 to 0.005 millimeter, silt.
Below 0.005 millimeter, clay.

In the following paragraphs are given the proportions of certain of the grain sizes found in the major soil textures:

Sands contain less than 20 per cent of silt and clay. (Include coarse, fine, and very fine sands.)

Sandy loams contain from 20 per cent to 50 per cent of silt and clay but do not have over 15 per cent of clay. (Include coarse, fine, and very fine sandy loams.)

Loams have more than 50 per cent of silt and clay combined but have less than 50 per cent of silt and less than 20 per cent of clay.

Silt loams have more than 50 per cent of silt and less than 20 per cent of clay.

Clay loams have more than 50 per cent of silt and clay combined but less than 50 per cent of silt and between 20 per cent and 30 per cent of clay. (Include sandy clay loams, clay loams, and silty clay loams.)

Clays have more than 50 per cent of silt and clay combined and more than 30 per cent of clay. (Include sandy clays and silty clays.)

In the field texture is determined by the feel of the soil mass when rubbed between the fingers. The following statements give the obvious physical characteristics of the basic textural groups:

Sand.—Sand is loose and granular. The individual grains can readily be seen or felt. Squeezed in the hand when dry it will fall apart when the pressure is released. Squeezed when moist, it will form a cast, but will crumble when touched.

Sandy loam.—A sandy loam is a soil containing much sand but having enough silt and clay to make it somewhat coherent. The individual sand grains can readily be seen and felt. Squeezed when dry, it will form a cast which will readily fall apart, but if squeezed when moist a cast can be formed that will bear careful handling without breaking.

Sands and sandy loams are classed as coarse, medium, fine, or very fine, depending on the proportion of the different sized particles that are present.

Loam.—A loam is a soil having a relatively even mixture of the different grades of sand and of silt and clay. It is mellow with a somewhat gritty feel, yet fairly smooth and slightly plastic. Squeezed when dry, it will form a cast that will bear careful handling, while the cast formed by squeezing the moist soil can be handled freely without breaking.

Silt loam.—A silt loam is a soil having a moderate amount of the fine grades of sand and only a small amount of clay, over half of the particles being of the size called "silt." When dry it may appear quite cloddy, but the lumps can be readily broken, and when pulverized it feels soft and floury. When wet the soil readily runs together and puddles. Either dry or moist it will form casts that can be freely handled without breaking. If squeezed between thumb and finger it will not "ribbon" but will give a broken appearance.

Clay loam.—A clay loam is a fine-textured soil which breaks into clods or lumps that are hard when dry. When the moist soil is pinched between the thumb and finger it will form a thin ribbon which will break readily, barely sustaining its own weight. The moist soil is plastic and will form a cast that will bear much handling. When kneaded in the hand it does not crumble readily but tends to work into a heavy compact mass.

Clay.—A clay is a fine-textured soil that forms very hard lumps or clods when dry. When the moist soil is pinched out between the thumb and fingers it will form a long, flexible ribbon.

Gravelly or stony soils.—All of the above classes of soils, if mixed with a considerable amount of gravel or stone, may be classed as gravelly sandy loams, gravelly clays, etc., as stony

sandy loams, stony loams, etc., or as sandy clay loams, sandy clays, etc.

Floury.—Fine-textured soil consisting predominantly of silt (or flocculated clay with aggregates of silt size) which when dry is incoherent, smooth, and dust-like.

Gritty.—Containing a sufficient amount of angular grains of coarse sand or fine gravel, so that these dominate the "feel." Usually applied to medium-textured soils (loams) where the actual quantity of these coarse grains is rather small.

Heavy (textured).—Applied to soils of fine texture in which clay predominates, with dense structure and firm to compact consistency. The term is also applied to soils containing a somewhat higher proportion of the finer separates than is typical of that textural class (as a "heavy sandy loam").

Light (textured).—Applied to soils of coarse to medium texture with very low silt and clay content, incoherent, single-grained structure, and loose consistency. The term is also applied to soils containing somewhat higher proportions of the coarser separates than is typical of that textural class (as a "light loam").

Sharp.—Containing angular particles in sufficient amount to dominate the "feel." Abrasive.

Smooth.—Containing well-rounded coarser particles and a predominance of the finer separates. Not abrasive.

COLOR

In order to recognize the same soil in some other locality, the color should be clearly stated. This statement should give the range of color included, for classification purposes. By range of color is meant such terms as black or dark brown, brown to reddish brown, etc. The color may indicate the drainage conditions under which the soil was formed and the chemical composition of the soil.

In using such terms as grayish brown, brownish gray, etc., the adjective is recognized as a modifying term. The grayish brown soil is a brown soil with a grayish cast sufficiently noticeable to require recognition; the brownish gray soil is a gray soil with a brown cast.

Other color terms are as follows:

Mottled.—The presence of spots, streaks, or splotches of one or more colors in a soil mass of another predominant color. In mottled soils the colors are not mixed and blended, but each is more or less distinct in the general ground color. In color descriptions the ground color and the color of the included spots should be designated. Mottling is usually but not necessarily associated with poor drainage. The use of the term should not be confined to poorly drained soils but should be applied wherever the term fits.

Marbled.—The presence of two or more distinct colors in approximately equal amounts not blended but more or less mixed in occurrence in the soil mass. In a marbled soil there is no general or predominant color, as in the case of a mottled soil.

Spotted, speckled, streaked, variegated.—Such terms can be used when their generally accepted meaning describes the color distributions that occur in the soils.

STRUCTURE

The term "soil structure" expresses the arrangement of the individual grains and aggregates that make up the soil mass. The structure may refer to the natural arrangement of the soil when in place and undisturbed (as structural profile) or to the soil at any degree of disturbance. The terms used indicate the character of the arrangement, the general shape and the size of the aggregates, and in some cases may indicate the consistency of those aggregates.

Adobe structure.—This term describes a soil which on drying cracks and breaks into irregular but roughly cubical blocks. The cracks are usually wide and deep and the blocks are from 20 to 50 or more centimeters across. (Adobe soils are usually heavy-textured and high in content of colloidal clay.)

Amorphous structure.—A soil of fine texture having a massive or uniform arrangement of particles throughout the horizon. Structureless. Found only in soils of finest texture, where individual grains can not be recognized.

Clod (or cloddy) structure.—Aggregates of irregular, angular shape, usually 4 centimeters or more in diameter and of a hard consistency.

Fine cloddy structure.—When most of the clods are close to the minimum size.

Coarse cloddy structure.—When most of the clods are 10 centimeters or more in diameter.

Columnar structure.—A natural arrangement of the soil mass in more or less regular columns separated by vertical cleavage lines, and usually broken by horizontal cracks into sections with longer vertical than horizontal axes, the tops of the columns being rounded.

Prismatic columnar structure.—Term used when the sections are very regular in size, straight-sided, with the vertical axes much longer than the horizontal axes and the tops of the columns flat.

Crumb structure.—Porous aggregates of irregular shape, rarely over 2 centimeters in diameter and of a medium to soft consistency.

Fine crumb structure.—Crumbs 5 millimeters or less in diameter.

Coarse crumb structure.—Crumbs 2 centimeters or more in diameter.

Crust (or crusted) structure.—This term is used where the upper or surface horizon coheres into plate or crust distinct from the horizon immediately below it.

Crust-mulch structure.—An arrangement where a surface crust is underlain by a horizon of loose, incoherent particles of mealy, crumb, or granular structure.

Fluffy structure.—A surface condition where the aggregates are loose, of light weight and fine texture, with no cohesion or evidence of arrangement; floury.

Dense structure.—Having a minimum of pore space and an absence of any large pores or cracks. Approaching amorphous.

Granular structure.—Aggregates varying in size to 2 centimeters in diameter, of medium consistency, and more or less subangular or rounded in shape.

Fine granular structure.—Aggregates under 5 millimeters diameter.

Coarse granular structure.—Aggregates close to maximum size.

Honeycomb structure.—A natural arrangement of the soil mass in more or less regular five or six sided sections separated by narrow or hairline cracks. Usually found as a surface structure or arrangement.

Hardpan.—An horizon of accumulation that has been thoroughly cemented to an indurated, rock-like layer that will not soften when wet. The term hardpan is not properly applied to hard clay layers that are not cemented, nor to those layers that may seem indurated when dry but which soften and lose their rock-like character when soaked in water. The true hardpan is cemented by materials that are not readily soluble, and is a hard layer that definitely and permanently (in nature) limits downward movement of roots and water.

Clay pan.—An horizon of accumulation or a stratum of stiff, compact, and relatively impervious clay. The clay pan is not cemented, and if immersed in water can be worked to a soft mass. Its presence may interfere with water movement or root development the same as a true hardpan. It is more difficult to overcome, for, whereas a hardpan can be shattered by explosives, the clay pan, after breaking by any means, will run together and re-form as soon as thoroughly wetted. The distinction between hardpan and clay pan is an important one in the soil classification.

Laminated structure.—An arrangement of the soil mass in very thin plates or layers, less than 1 millimeter in thickness, lying horizontal or parallel to the soil surface. Usually medium to soft consistency.

Massive structure.—A soil mass showing no evidence of any distinct arrangement of the soil particles. Structureless. May be found in soils of any structure.

Mealy structure.—A crumb-like structure in which the aggregates are of soft to very soft consistency and usually less than 5 millimeters in diameter.

Nut structure.—Compact aggregates, more or less rounded in shape, of hard to medium consistency, and from one-half to 4 centimeters in diameter.

Fine nut structure.—Aggregates below 1 centimeter in diameter.

Coarse nut structure.—Aggregates over 3 centimeters in diameter.

Plate (or platy) structure.—An arrangement of the soil mass in plates or layers 1 to 5 millimeters or more in thickness, lying horizontal or parallel to the soil surface. Usually medium to hard consistency.

Single-grained structure.—An incoherent condition of the soil mass with no arrangement of the individual particles into aggregates. Structureless. Usually found in soils of coarse texture.

Structureless.—Without any discernible structure or arrangement of the soil particles into aggregates. This condition is better expressed by the terms single-grained, massive, amorphous, etc.

Vesicular structure.—A soil horizon or soil aggregate containing many small rounded cavities smooth on the inside as though formed by gas bubbles.

CONSISTENCY

"Soil consistency" is a term expressing the degree of cohesion of the soil particles and the resistance offered to forces tending to deform or rupture the aggregates. Consistency and structure

are closely related and frequently interdependent. The terms expressing consistency and structure are distinct, however, and need not be confused or used with double meaning. A study of published reports shows a general use of terms expressing both the consistency and the structure in nearly all soil descriptions.

Brittle.—A soil which when dry will break with a sharp, clean fracture. If struck a sharp blow, it will shatter into cleanly broken hard fragments.

Mellow.—Soil particles or aggregates are weakly adhered in a rather porous mass, readily yielding to forces causing rupture. A consistency softer than friable. Without tendency to pack.

Plastic.—Readily deformed without rupture. Pliable but cohesive. Can be readily molded. Puttylike. This term applies to those soils in which at certain stages of moisture the grains will readily slip over each other without the mass cracking or breaking apart.

Soft.—Yielding readily to any force causing rupture or deformation. Aggregates readily crushed between fingers.

Sticky.—Applied to soils showing a decided tendency when wet to adhere to other materials and foreign objects.

Firm.—Resistant to forces tending to produce rupture or deformation. Moderately hard. Aggregates can be broken between fingers.

Friable.—Aggregates readily ruptured and crushed with application of moderate force. Easily pulverized or reduced to crumb or granular structure.

Hard.—Resistant to forces tending to cause rupture or deformation. Difficult or impossible to crush aggregates with fingers only.

Tenacious.—Soils showing a decided resistance to rupture. Soil mass adheres firmly.

The terms "sticky" and "tenacious" are often used as synonyms, but in soil usage the former is taken to refer to adhesion, the latter to cohesion. Both terms may be applicable to a soil at the same time.

Stiff.—Resistant to rupture or deformation. A soil stratum or horizon that is firm and tenacious, and tending toward imperviousness. Usually applied to condition of the soil in place and when moderately wet.

Tight.—A stratum or horizon that is compact, impervious and tenacious, and usually plastic.

Tough.—Resistant to rupture. Tenacious. A stratum or horizon that can be readily bored into with the auger but which requires much force in breaking loose and pulling out the core of soil.

COMPACTNESS

Compactness is the degree of resistance offered by a soil to the penetration of a pointed instrument.

Impervious.—Highly resistant to penetration by water and usually resistant to penetration by air and plant roots. Impenetrable. In field practice the term is applied to strata or horizons that are very slowly penetrated by water and that retard or restrict root penetration.

Indurated.—(See under cementation).

Loose.—Soil particles or small aggregates are independent of each other or cohere very weakly with a maximum of pore space and a minimum resistance to forces tending to cause rupture.

Cheesy.—Having a more or less elastic character, deforming considerably without rupture, yet broken without difficulty or the application of much force. (Characteristic of certain highly colloidal soils when thoroughly wet.)

Compact.—The soil packed together in a dense, firm mass, but without any cementation. Noticeably resistant to forces tending to cause rupture or deformation. Coherent. Hard.

Relative degree of compactness may be expressed by terms as slightly compact, very compact, etc.

CEMENTATION

Cementation.—A condition occurring when the soil grains or aggregates are caused to adhere firmly and are bound together by some material that acts as a cementing agent (as colloidal clay, iron or aluminum hydrates, lime carbonate, etc.).

The degree of cementation or the persistence of the cementation when the soil is wetted should be stated. Some terms indicate the permanence, as "indurated," "hardpan," etc.

Firmly cemented.—Cementing material of considerable strength requiring considerable force to rupture the mass. Usually breaks, with clean though irregular fractures, into hard fragments.

Indurated.—Cemented into a very hard mass which will not soften or lose its firmness when wet, and which requires much force to cause breakage. Rock-like.

Weakly cemented.—Term applied when cementing material is not strong, and the aggregates can be readily broken into fragments with a more or less clean fracture.

Softly cemented.—Term applied when cementing material is not strong nor evenly diffused throughout the mass. Aggregates are readily crushed, but do not break with a clean fracture.

CHEMICAL COMPOSITION

Peat soil.—Composed predominantly of organic material, highly fibrous, with easily recognized plant remains.

Muck soil.—Composed of thoroughly decomposed black organic material, with a considerable amount of mineral soil material, finely divided and with a few fibrous remains.

Leaf mold.—The accumulation on the soil surface of more or less decomposed organic remains, usually the leaves of trees and remains of herbaceous plants. The A horizon.

Alkaline soil.—A soil containing an excessive amount of the alkaline (in true chemical sense) salts.

Saline soil.—A soil containing excessive amounts of the neutral or non-alkaline salts.

Calcareous soil.—A soil containing sufficient calcium carbonate to effervesce when tested with weak (N/10) hydrochloric acid. Depending on the amounts present, these soils may be designated as slightly calcareous, strongly calcareous, etc.

Acid soil.—A soil which is deficient in available bases, particularly calcium, and which gives an acid reaction when tested by standard methods. Field tests are made by the use of litmus, of Soiltex, and of other indicators. There is no full agreement on the most satisfactory test for acidity or as to the actual character of an acid soil. The intensity or degree of acidity may be expressed by qualifying words, "strongly," "moderately," etc.

APPENDIX B

SAMPLE REPORT OF A SUBGRADE SURVEY OF A ROAD IN SERVICE

ROAD DESCRIPTION

Designation.—United States Highway No. 51, Federal aid project 82–A, situated two miles south of Jackson, Hinds County, Miss.

Date constructed.—1927.

Width.—Roadway, 26 feet; pavement, 18 feet.

Pavement thickness.—Seven inches at the edges and 6 inches at the center, stations 97+00 to 108+88; 9 inches at the edges and 6 inches at the center, stations 108+88 to 112+50.

Joints.—One-half inch transverse expansion joints spaced about 30 feet apart, stations 97+00 to 108+88; tongue-and-groove longitudinal center joint with metal center strip and ½-inch transverse expansion joints spaced about 50 feet apart, stations 108+88 to 112+50.

Reinforcement.—Welded fabric placed 2 inches from the top of the slab.

Concrete design.—One part Portland cement, 2 parts local sand, and 3 parts local gravel.

Curing.—Two per cent calcium chloride admixture.

Compressive strength of concrete.—Average of sixteen 6 by 12 inch cylinders tested at the end of 28 days equaled 3,230 pounds per square inch.

Drainage.—Eight-inch bell-and-spigot concrete drain placed in west ditch at a depth of 3 feet below subgrade elevation. Trench is back filled with washed gravel as shown in Figure 1.

Reasons for survey.—(*a*) To determine if the detrimental distortion suffered by an appreciable number of the pavement slabs comprising this project was related to the subgrade, and (*b*) to determine what precautions might be taken to prevent this type of failure from occurring in pavements laid on similar subgrades in the future.

Cooperating agencies.—The Mississippi State Highway Department and the Portland Cement Association.

DATA FURNISHED BY THE SURVEY

The soil profile.—The profile consists of five layers designated as layer 1, layer 3, layer 5, layer 6, and layer 7, shown in Figure 1.

Layer 1, averaging about 18 inches in thickness, consists of a light brown to reddish or grayish brown mellow silt loam of fine crumb structure containing both black and brown iron concretions. The grayish brown color becomes rather pronounced in the lower 10 inches of layer 1 where it grades into layer 3.

Layer 1 soil is friable when dry, has a pasty consistency when wet, and according to Table 2, it has characteristics similar to those of the plastic varieties of the group A–4 subgrades.

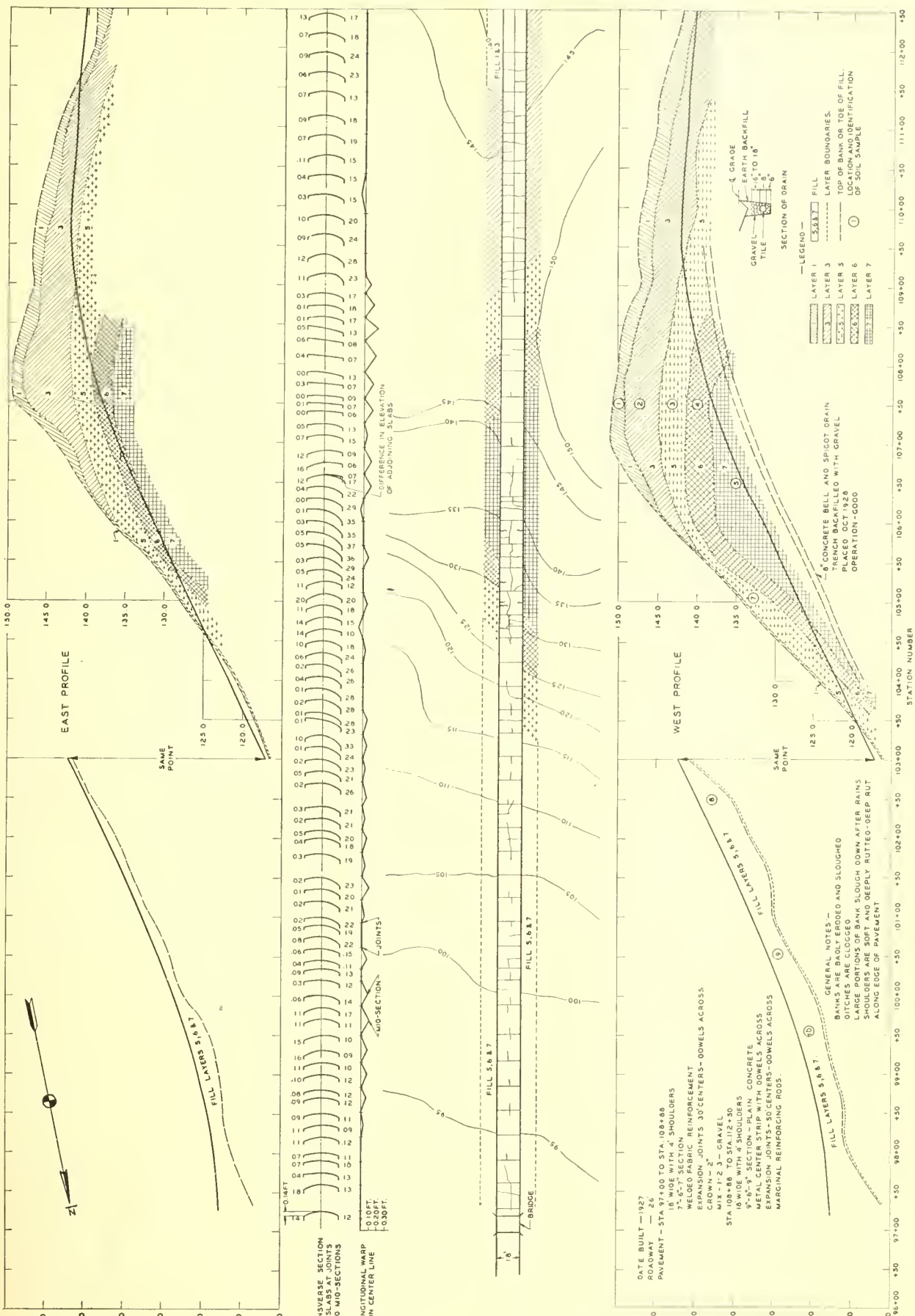


FIGURE 1.—SUBGRADE SURVEY SHEET FOR ROAD IN SERVICE

Layer 3 grades from layer 1 into a brown and gray mottled silty clay loam or silty clay of moderately compact structure. The upper 10 inches of layer 3 is a transition from layer 1, but is much more dense and compact. This compactness increases with increasing depth until at about 5½ feet layer 3 has a higher moisture content and grades into the angular-structured layer 5.

The soil of layer 3 is friable when dry and very plastic when wet and is permeable regardless of its compactness. Bank erosion, shown in Figure 2, causes the soil of layer 3 to be fissured and to assume the form of pinnacles upon drying. According to Table 2, layer 3 consists of a more plastic variety of the Group A–4 subgrade than layer 1.

Layer 3, like layer 1, contains dark-brown and black concretions in large amounts.

FIGURE 2.—EROSION OF BACK SLOPES IS VERY IRREGULAR AND THERE IS CONSIDERABLE SLOUGHING OF MATERIAL INTO DITCHES

TABLE 1. *Mechanical analyses of soils found in soil profile*

Identification No.	Layer	Percentage of particles having diameters smaller than—					
		2 mm.	0.5 mm.	0.25 mm.	0.05 mm.	0.005 mm.	0.001 mm.
1	1	100	98	97	86	23	12
2	3	100	97	95	93	27	15
3	5	98	92	89	89	36	16
4	6	100	100	99	81	43	17
5	7	100	100	97	93	72	47
7	5	100	100	99	98	74	[1]
8	[2] 5, 6, 7	100	100	99	92	[1]	----
9	[2] 5, 6, 7	100	98	97	91	[1]	----
10	[2] 5, 6, 7	100	98	97	90	[1]	----

[1] Flocculated. [2] Fill.

TABLE 2.—*Physical characteristics of particles passing the 0.5 millimeter sieve*

Identification No.	Layer	Liquid limit	Plastic index	Shrinkage		Moisture equivalent		Group
				Limit	Ratio	Centrifuge	Field	
		Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*	
1	1	39	17	20	1.8	31	31	A–4
2	3	45	22	22	1.7	37	35	A–4
3	5	50	33	12	2.0	[1] 50	34	A–6
4	6	85	51	12	2.0	[1] 98	52	A–7
5	7	112	77	11	2.0	[1] 125	71	A–7
7	5	98	64	12	2.0	[1] 98	59	A–7
8	[2] 5, 6, 7	102	67	12	1.9	[1] 101	70	A–7
9	[2] 5, 6, 7	104	72	13	2.0	[1] 92	60	A–7
10	[2] 5, 6, 7	76	44	12	1.9	[1] 95	59	A–7

[1] Water-logged. [2] Fill.

Layer 5 consists of brown and gray mottled heavy, plastic, sticky clay, with well-defined particles of angular structure about one-eighth inch in diameter or larger, which have a wet, shiny, and slick surface. This layer is very much wetter than overlying zones, is open-structured, and permits the movement of water. In some portions of layer 5 bright red mottlings are found. According to Table 2, the soil of this layer has characteristics common to the highly plastic varieties of the Group A–6 subgrades (sample 3), but when calcium compounds are present in appreciable amounts (sample 7), it exhibits characteristics similar to those of the Group A–7 subgrades.

Layer 6 grades from layer 5, at a depth of about 9 feet below the ground surface, into a very sticky plastic clay, bluish gray in color, mottled with brown. As shown in Figure 3 it exists in the face of cuts as slick and shiny angular particles or clods, ¼ inch to 2 inches in the longest dimension. These clods, the larger of which are dense and have a fibrous structure, are similar to putty in consistency and are easily crushed. Water moves through this layer at a rate much slower than it moves through layer 5. The characteristics of the soil of this layer are similar to those of the Group A–7 subgrades.

Layer 7, at a depth of about 10 or 11 feet below the ground surface, grades gradually from layer 6 into a bluish gray, sticky, plastic and very dense indurated clay containing both black and brown stains. In cut faces this material, similar to that of layer 6, exists as clods or chunks, fibrous in structure and easily crushed. The soil of layer 7 is impervious, holds absorbed water tenaciously, shrinks in appreciable amount when dried, slakes readily in the presence of water and exhibits characteristics generally indicative of the Group A–7 subgrades.

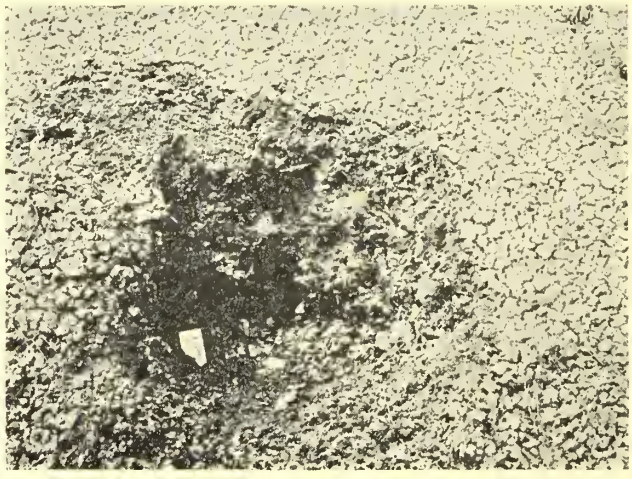


FIGURE 3.—RIGHT BANK AT STATION 105+16 SHOWING STRUCTURE OF LAYER 6 MATERIAL, AND SLOUGHED MATERIAL ON SURFACE

The soils of layers 5, 6, and 7 effervesce strongly when treated with dilute hydrochloric acid, thus indicating the presence of lime in appreciable amount.

Condition of shoulders and banks.—The shoulders are very soft and badly rutted as shown in Figure 4, at those locations where the road traverses the clays of layers 5, 6, and 7. In the same locations the back slopes are badly eroded (see Figure 2) and after rains are apt to slide and clog the side ditches.

Condition of pavement.—Typical pavement condition with respect to both slab distortion and cracking is disclosed by Figure 5 and Tables 3, 4, and 5.

The longitudinal warp referred to in Table 3 is defined as the difference between the average elevation of the ends of the slab and the elevation at a point midway between the ends of the slab measured at the center line. Corrections were made for vertical curvature.

The transverse warp referred to in Table 4 is defined as the present crown of the pavement at the transverse joints subtracted from the original which, exclusive of superelevated curves, was assumed to be 2 inches.

Warp, either transverse or longitudinal less than 0.03 ft. is neither shown on Figure 1 nor included in the computation of the averages in Tables 3 and 4. The apparent bumps appearing in the pavement at the transverse joints and caused by the distortion of the slabs longitudinally is illustrated in Figure 4. An analysis of the longitudinal and transverse cracking is given in Table 5.

TABLE 3.—*Longitudinal warp on center line*

Layer	Length	Total warp	Maximum warp	Total slabs	Number of slabs warped	Percentage of slabs warped	Average [1] warp per slab	Average [2] warp per slab
	Feet	*Feet*	*Feet*				*Feet*	*Feet*
5, 6, 7	497	1.50	0.18	16	13	81	0.12	0.09
5, 6, 7, fill	664	.97	.10	21	16	76	.06	.05
3 [3]	311	.04	.04	6	1	17	.04	.01

[1] Based on number of slabs warped.
[2] Based on total number of slabs.
[3] Center-joint section.

TABLE 4.—*Transverse warp from center line*

Layer	Number of joints	Total warp	Maximum warp	Average warp
		Feet	*Feet*	*Feet*
5, 6, 7	16	3.03	0.23	0.19
5, 6, 7, fill	23	2.78	.15	.12
3 [1]	5	.64	.13	.13

[1] Center-joint section.

TABLE 5.—*Longitudinal and transverse cracking*

Layer	Length	Number of joints	Transverse				Londitudinal		
			Number	Length	Length per slab	Average slab length	Length	Per cent	Length per slab
	Feet			*Feet*	*Feet*	*Feet*	*Feet*		*Feet*
5, 6, 7	497	16	8	147	9	21	225	45	14
5, 6, 7, fill	650	21	5	89	4	25	269	41	13
3 [1]	311	6	9	162	27	21			
5 [1]	51	1	3	58	58	13			

[1] Center-joint section.

FIGURE 4.—SHOULDERS ARE RUTTED TO A DEPTH OF 4 INCHES BELOW BOTTOM OF SLAB

SUMMARY

The results obtained from the survey may be summarized as follows:

1. Distortion both longitudinal and transverse occurred in greatest amount in the pavement slabs 18 feet wide and about 30 feet long laid on soil layers 5, 6, and 7 when in cut, and in but slightly less amount in similar slabs laid on mixtures of layers 5, 6, and 7 used in fills.

2. The slabs containing a center joint and transverse joints spaced about 50 feet apart laid primarily on layers 1 and 3 suffered warping in very much less amount than the slabs about 30 feet long laid on layers 5, 6, and 7.

3. The slabs of another pavement (18 feet wide and about 30 feet long) separated from layers 5, 6, and 7 of these soils by several feet of good top soil did not suffer distortion in detrimental amount.

4. Longitudinal cracks occurred only in those slabs constructed without center joint, and were either short cracks beginning at expansion joints or cracks extending throughout the lengths of the slabs. In every case, however, the longitudinal cracks were situated above dowels extending across the transverse joints. In many instances a longitudinal crack was found over one dowel on one side of a joint and over the next dowel on the other side of the joint.

FIGURE 5.—WARPED SLABS. TRANSVERSE JOINTS ARE MARKED WITH STICKS

5. More transverse cracks occured in the slabs about 50 feet long than in the slabs 30 feet long. The greatest concentration of combined longitudinal and transverse cracking occured in the slabs without center joint laid in cut on a subgrade consisting of either layer 7 or a combination of layers 5, 6, and 7. (See station 104+50 to station 106+50, Figure 1.)

6. Longitudinal cracking was not important in the slabs which warped in greatest amount. Even at several joints where the slabs had faulted in amounts equal to as much as 2 inches no cracks occurred above the dowels.

7. Neither the longitudinal nor the transverse cracks were of appreciable width. This fact indicates that the mesh reinforcement has served to prevent the separation of the cracked fragments of slabs.

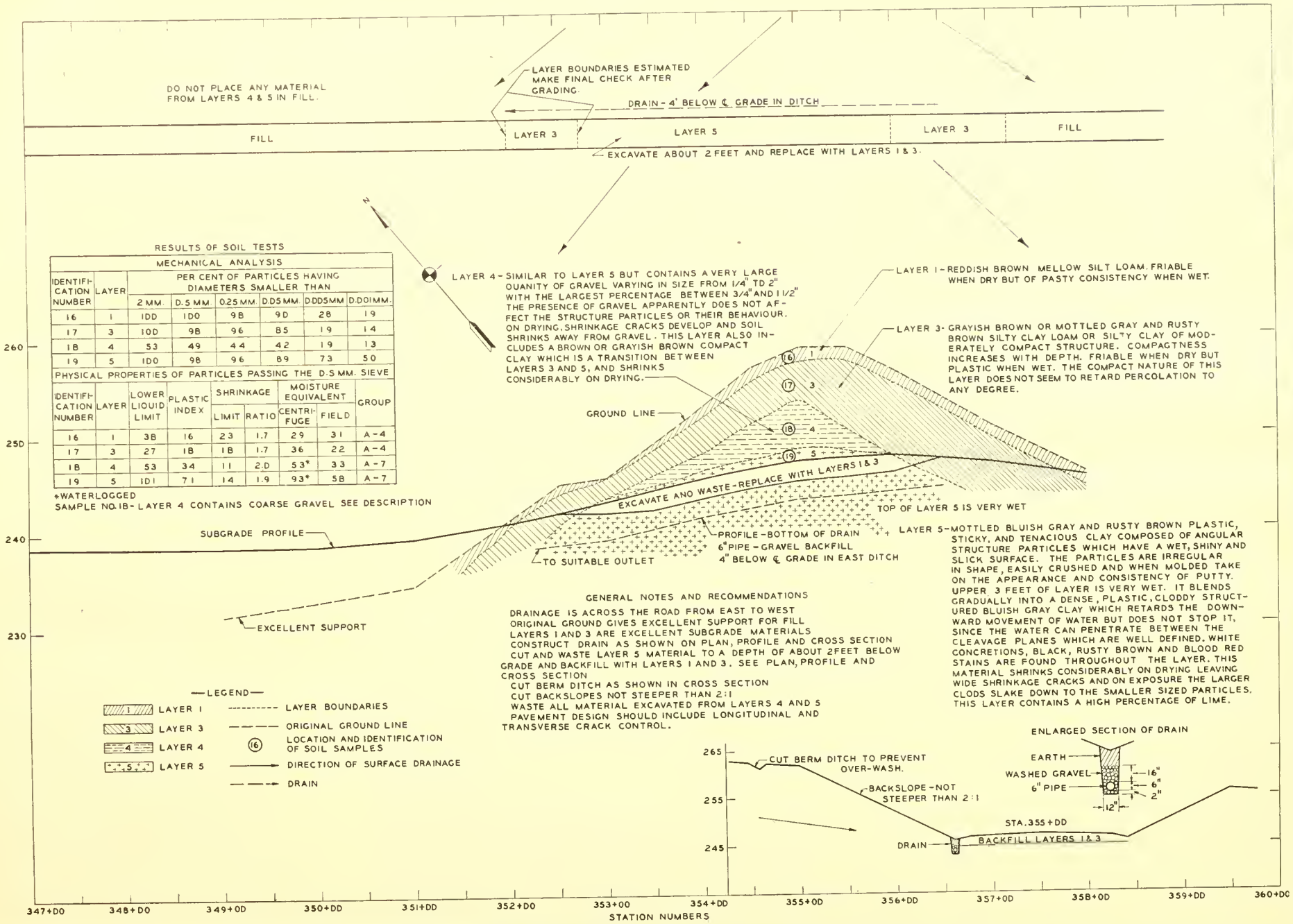
8. Generally the subgrade soil was found to be in a very soft and wet condition. Frequently when the filler was removed from the expansion joints water rose quickly to the surface of the pavement.

9. The drain serves to carry a large flow of water for several days after rains, but does not eliminate the slab movements or prevent the subgrade from becoming wet.

10. The sliding of layers, 5, 6 and 7 when occurring in the face of cuts constitutes a serious maintenance problem. In this connection a scrutiny of records covering a period of 50 years, which were furnished by a railway company operating in this region, indicates that when used in fills the soils of these layers are likely to prove troublesome until the slope becomes approximately 1:5.

SUBGRADE SURVEY SHEET GIVING INFORMATION FOR THE DESIGN OF NEW ROADS

Figure 6 is an example of the final form of survey sheet submitted to the engineer in charge of design. This form was prepared from a survey of approximately 14 miles of graded highway in Mississippi. The recommendations are based on data furnished by the behavior of existing pavements on similar subgrades. The location, grades and type of surfacing had already been established on this highway.


DO NOT PLACE ANY MATERIAL FROM LAYERS 4 & 5 IN FILL.
LAYER BOUNDARIES ESTIMATED MAKE FINAL CHECK AFTER GRADING.
DRAIN - 4' BELOW ℄ GRADE IN DITCH
FILL
LAYER 3
LAYER 5
LAYER 3
FILL
EXCAVATE ABOUT 2 FEET AND REPLACE WITH LAYERS 1 & 3.
LAYER 4 - SIMILAR TO LAYER 5 BUT CONTAINS A VERY LARGE QUANITY OF GRAVEL VARYING IN SIZE FROM 1/4" TO 2" WITH THE LARGEST PERCENTAGE BETWEEN 3/4" AND 1 1/2" THE PRESENCE OF GRAVEL APPARENTLY DOES NOT AFFECT THE STRUCTURE PARTICLES OR THEIR BEHAVIOUR. ON DRYING, SHRINKAGE CRACKS DEVELOP AND SOIL SHRINKS AWAY FROM GRAVEL. THIS LAYER ALSO INCLUDES A BROWN OR GRAYISH BROWN COMPACT CLAY WHICH IS A TRANSITION BETWEEN LAYERS 3 AND 5, AND SHRINKS CONSIDERABLY ON DRYING.
LAYER 1 - REDDISH BROWN MELLOW SILT LOAM. FRIABLE WHEN DRY BUT OF PASTY CONSISTENCY WHEN WET.
LAYER 3 - GRAYISH BROWN OR MOTTLED GRAY AND RUSTY BROWN SILTY CLAY LOAM OR SILTY CLAY OF MODERATELY COMPACT STRUCTURE. COMPACTNESS INCREASES WITH DEPTH. FRIABLE WHEN DRY BUT PLASTIC WHEN WET. THE COMPACT NATURE OF THIS LAYER DOES NOT SEEM TO RETARD PERCOLATION TO ANY DEGREE.
GROUND LINE
EXCAVATE AND WASTE - REPLACE WITH LAYERS 1 & 3
TOP OF LAYER 5 IS VERY WET
SUBGRADE PROFILE
PROFILE - BOTTOM OF DRAIN
6" PIPE - GRAVEL BACKFILL
4" BELOW ℄ GRADE IN EAST DITCH
TO SUITABLE OUTLET
LAYER 5 - MOTTLED BLUISH GRAY AND RUSTY BROWN PLASTIC, STICKY, AND TENACIOUS CLAY COMPOSED OF ANGULAR STRUCTURE PARTICLES WHICH HAVE A WET, SHINY AND SLICK SURFACE. THE PARTICLES ARE IRREGULAR IN SHAPE, EASILY CRUSHED AND WHEN MOLDED TAKE ON THE APPEARANCE AND CONSISTENCY OF PUTTY. UPPER 3 FEET OF LAYER IS VERY WET. IT BLENDS GRADUALLY INTO A DENSE, PLASTIC, CLODDY STRUCTURED BLUISH GRAY CLAY WHICH RETARDS THE DOWNWARD MOVEMENT OF WATER BUT DOES NOT STOP IT, SINCE THE WATER CAN PENETRATE BETWEEN THE CLEAVAGE PLANES WHICH ARE WELL DEFINED. WHITE CONCRETIONS, BLACK, RUSTY BROWN AND BLOOD RED STAINS ARE FOUND THROUGHOUT THE LAYER. THIS MATERIAL SHRINKS CONSIDERABLY ON DRYING LEAVING WIDE SHRINKAGE CRACKS AND ON EXPOSURE THE LARGER CLODS SLAKE DOWN TO THE SMALLER SIZED PARTICLES. THIS LAYER CONTAINS A HIGH PERCENTAGE OF LIME.
EXCELLENT SUPPORT
260
250
240
230
GENERAL NOTES AND RECOMMENDATIONS
DRAINAGE IS ACROSS THE ROAD FROM EAST TO WEST
ORIGINAL GROUND GIVES EXCELLENT SUPPORT FOR FILL
LAYERS 1 AND 3 ARE EXCELLENT SUBGRADE MATERIALS
CONSTRUCT DRAIN AS SHOWN ON PLAN, PROFILE AND CROSS SECTION
CUT AND WASTE LAYER 5 MATERIAL TO A DEPTH OF ABOUT 2 FEET BELOW GRADE AND BACKFILL WITH LAYERS 1 AND 3. SEE PLAN, PROFILE AND CROSS SECTION
CUT BERM DITCH AS SHOWN IN CROSS SECTION
CUT BACKSLOPES NOT STEEPER THAN 2:1
WASTE ALL MATERIAL EXCAVATED FROM LAYERS 4 AND 5
PAVEMENT DESIGN SHOULD INCLUDE LONGITUDINAL AND TRANSVERSE CRACK CONTROL.
—LEGEND—
LAYER 1
LAYER 3
LAYER 4
LAYER 5
LAYER BOUNDARIES
ORIGINAL GROUND LINE
LOCATION AND IDENTIFICATION OF SOIL SAMPLES
DIRECTION OF SURFACE DRAINAGE
DRAIN
ENLARGED SECTION OF DRAIN
EARTH
WASHED GRAVEL
6" PIPE
16"
6"
2"
12"
265
255
245
CUT BERM DITCH TO PREVENT OVER-WASH.
BACKSLOPE - NOT STEEPER THAN 2:1
STA. 355+DD
DRAIN
BACKFILL LAYERS 1 & 3
347+DO
348+DO
349+OD
350+DD
351+DD
352+DO
353+00
354+DD
355+OD
356+DD
357+OD
358+OD
359+DD
360+DC
STATION NUMBERS

RESULTS OF SOIL TESTS

MECHANICAL ANALYSIS

IDENTIFICATION NUMBER	LAYER	PER CENT OF PARTICLES HAVING DIAMETERS SMALLER THAN					
		2 MM.	D.5 MM.	0.25 MM.	D.D5 MM.	D.DD5 MM.	D.DI MM.
16	1	1DD	1D0	98	9D	28	19
17	3	10D	98	96	85	19	14
18	4	53	49	44	42	19	13
19	5	1D0	98	96	89	73	50

PHYSICAL PROPERTIES OF PARTICLES PASSING THE D.5 MM. SIEVE

IDENTIFICATION NUMBER	LAYER	LOWER LIQUID LIMIT	PLASTIC INDEX	SHRINKAGE		MOISTURE EQUIVALENT		GROUP
				LIMIT	RATIO	CENTRIFUGE	FIELD	
16	1	38	16	23	1.7	29	31	A-4
17	3	27	18	18	1.7	36	22	A-4
18	4	53	34	11	2.D	53*	33	A-7
19	5	1D1	71	14	1.9	93*	58	A-7

*WATERLOGGED
SAMPLE NO. 18 - LAYER 4 CONTAINS COARSE GRAVEL SEE DESCRIPTION

FIGURE 6.—SUBGRADE SURVEY SHEET FOR THE DESIGN OF NEW ROAD

SOME OBSERVATIONS ON THE MODULUS OF RUPTURE OF FROZEN CONCRETE BEAMS

Reported by Andrew P. Anderson, Highway Engineer, Division of Management, U. S. Bureau of Public Roads

NEAR the close of some extensive field studies of efficiency in concrete road construction during the fall of 1929, the temperature dropped below 20° F., with the result that four regular 6 by 8 inch concrete beams were apparently frozen through when they were broken 24 hours after they had been removed from the curing tank. A long series of exactly similar beams had previously been broken on this same job during the course of several months. The indicated modulus of rupture of these frozen beams, in comparison with those not frozen, was so large that when an opportunity arose in the summer of 1930 to make some further and better controlled observations on the modulus of rupture of frozen beams it was at once accepted.

It happened that a production study was being conducted on the construction of a concrete road which passed directly by an ice-making plant with facilities available for storing a considerable number of beams and cylinders at a temperature never higher than 28° F., and at night as low as 25° F. Seventeen standard 6 by 8 by 36 inch beams, comprising five sets of 3 beams each and one 2-beam set, were therefore prepared at once. These beams were all made from the concrete regularly used on the job and the samples for each set of beams were obtained in exactly the same manner as were the regular test beams which were made daily for controlling the strength and quality of the concrete, and for determining when the pavement could be opened to traffic. Each set of three beams for the freezing test was made from one batch of concrete as it was dumped over the molds placed side by side on the subgrade, and in all but one case two beams were sooner or later placed in the cold room. In every case one beam from each set was kept as a control and cured 24 hours along the roadside under damp burlap and then stored under water until broken on the 28th day.

Since the work was carried on during midsummer, considerable care was necessary to prevent thawing during the flexure tests. The beams were wrapped in ice and burlap; and the clamps of the breaking machine and the top of the beam were kept covered with ice during the progress of the breaking. Some thawing probably did occur, especially on the first beams tested, before the completion of the second break. This is indicated by the fact that the second break on several of the frozen beams gave considerably lower results than the first break. For the beams broken during the 1930 job (job B, Table 1) the first break probably more nearly represents the condition of a thoroughly frozen beam.

Figure 1 shows clearly how each of the 17 beams of job B was treated in curing, and the length of time of each treatment. The modulus of rupture in pounds per square inch for each break is given in Table 1. The two sets of job A are those frozen during the 1929 job. They were removed from the water on the 27th day; and as the outside temperature during the night fell to about 12° F., these beams, which were stored in an unheated frame structure, were apparently frozen through when they were broken the following day. The third beam in each of these two sets was not made from the same batch, but is a representative beam from those broken a few days before the freeze occurred. All the beams on job A were broken on a multiple-lever field beam-breaking machine which had been used by the same men in breaking a large number of beams during the previous three months. The beams on job B also were all broken by the same men, but on a simple cantilever apparatus which gave very consistent results, of practically the same value as those given by the standard two-point loading in a universal testing machine. A good grade of gravel aggregate was used on both jobs. Job A was designed to give a modulus of rupture of about 800 pounds per square inch as compared with about 550 pounds for job B. The proportions by volume were 1:1.55:3.50 for job A and 1:2.20:3.98 for job B.

TABLE 1.—*Results of modulus of rupture tests of frozen concrete beams and control specimens not subjected to freezing*

Set No.	Beam No.	Modulus of rupture in pounds per square inch			
		First break	Second break	Third break	Average
JOB A					
1	1	1,152	1,172		1,162
	2	969	1,014		992
	3	843	861		852
2	1	993	1,046		1,019
	2	1,072	1,046		1,059
	3	825	782		803
JOB B					
1	1	1,010	824		917
	2	558	560	511	543
2	1	962	793		877
	2	273	362		315
	3	556	561	601	575
3	1	1,185	1,005		1,095
	2	545	446	504	498
	3	510	503	528	514
4	1	665	618	757	680
	2	501	430	402	444
	3	506	482	493	494
5	1	428	510	475	471
	2	257	307		282
	3	597	549		573
6	1	440	459	434	444
	2	404	445	433	427
	3	562	528		545

The apparent strength gained by the frozen beams is very striking—in some cases, more than 100 per cent above the corresponding control beam (job B, set 3). In but one case (job A, set 1, beam 2) is the apparent gain in strength imparted by complete freezing less than 25 per cent of the strength of the corresponding unfrozen beam. One of the beams (job B, set 2, No. 1) which was placed in the cold room but three hours after it was made and before it had attained its final set gave a surprisingly high figure for both breaks—962 and

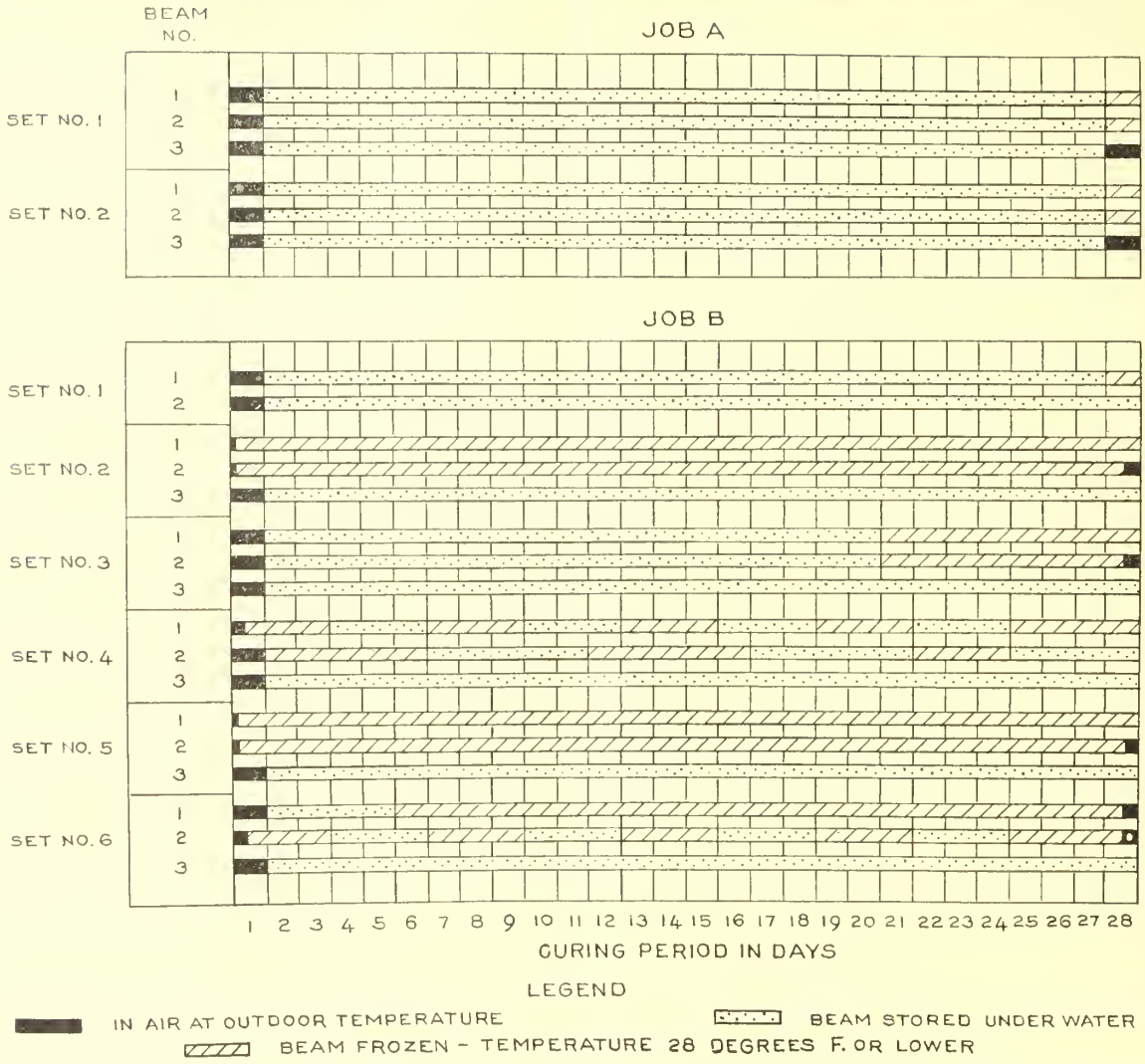

BEAM NO.
JOB A
SET NO. 1
SET NO. 2
JOB B
SET NO. 1
SET NO. 2
SET NO. 3
SET NO. 4
SET NO. 5
SET NO. 6
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28
CURING PERIOD IN DAYS
LEGEND
IN AIR AT OUTDOOR TEMPERATURE
BEAM STORED UNDER WATER
BEAM FROZEN - TEMPERATURE 28 DEGREES F. OR LOWER

FIGURE 1.—DIAGRAM SHOWING CURING TREATMENT OF CONCRETE BEAMS SUBJECTED TO FREEZING AND CORRESPONDING CONTROL SPECIMENS

793 pounds per square inch. The contrast with the companion beam which was treated in exactly the same manner, except that it was taken from the cold room and left in the open summer air for about six hours before it was broken, is most striking, for the latter beam gave moduli of only 273 and 362 pounds per square inch for the two breaks. This would seem to indicate rather clearly that the concrete gained but little actual strength while stored in the cold room. On the other hand, beams 2 of job B, sets 4 and 6, apparently suffered only a moderate reduction in modulus of rupture from being alternately frozen and thawed after they had attained their final set. These beams were placed in the cold room 24 and 7 hours, respectively, after they were molded. Beam 1 of set 5, job B was stored at some distance from the freezing coils and appeared to be only partly frozen when broken.

From these data it would appear that during cold weather a reasonable amount of care should be exercised in breaking any field beams which may form a criterion for establishing the time when the pavement can be thrown open to traffic. Unless the beams have been well protected, they should not be broken except at times when the air temperature has been well above the freezing point for at least five or six hours. A temperature of only 5° or 6° F. below the freezing point, if of sufficient duration, might readily freeze an exposed green concrete beam sufficiently to indicate a modulus of rupture for the frozen beam 50 per cent or more above what the same beam would give if broken when thawed. An otherwise properly cured beam will show a large increase in apparent strength if frozen immediately prior to breaking. This fact is shown by both sets of job A and set 1 of job B. These beams were properly cured until 24 hours prior to breaking. It is practically certain that the beam of job B, set 1, was frozen through, but it is not equally certain that this was the case with the beams of job A, since the temperature within the building in which the beams were stored may have been several degrees higher than the recorded outdoor temperature. To all appearances, however, the beams were frozen through.

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Applicants are urgently rquested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.
Report of the Chief of the Bureau of Public Roads, 1929.
Report of the Chief of the Bureau of Public Roads, 1930.

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*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.

*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.

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109M. Federal Legislation and Regulations Relating to the Improvement of Federal-Aid Roads and National-Forest Roads and Trails, Flood Relief, and Miscellaneous Matters.

MISCELLANEOUS PUBLICATIONS

No. 76MP. The Results of Physical Tests of Road-Building Rock.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio. (1927)

Report of a Survey of Transportation on the State Highways of Vermont. (1927)

Report of a Survey of Transportation on the State Highways of New Hampshire. (1927)

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio. (1928)

Report of a Survey of Transportation on the State Highways of Pennsylvania. (1928)

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.

Vol. 5, No. 19, D-3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.

Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.

Vol. 11, No. 10, D 15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

AUGUST 31, 1931

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal-aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total	Estimated total cost	Federal-aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total		
Alabama	2,133.2	$ 6,845,545.28	$ 3,387,188.08	231.2	104.6	335.8	$	$				$ 3,306,188.88	Alabama
Arizona	885.3	5,735,381.77	3,930,790.43	258.0	189.8	447.9						313,320.07	Arizona
Arkansas	1,786.8	7,402,833.56	3,964,135.82	175.1	112.8	287.7	869,006.72	434,503.34	24.2	15.7	39.9	77,186.05	Arkansas
California	1,962.6	12,621,031.84	5,198,777.55	316.1	110.9	426.8	1,742,651.42	733,148.72	54.8		54.8	159,422.84	California
Colorado	1,350.1	6,520,684.38	3,464,439.46	248.1	96.9	346.0	1,737,485.18	916,874.81	91.7	17.5	109.2	545,403.58	Colorado
Connecticut	263.4	4,878,672.93	1,915,984.63	47.9		47.9							Connecticut
Delaware	310.4	1,114,638.43	556,564.71	55.5		55.5						73,518.71	Delaware
Florida	554.4	6,161,508.65	2,872,466.15	174.4		174.4						1,609,098.91	Florida
Georgia	2,835.4	8,855,198.24	4,214,619.21	298.8	121.4	420.2	2,731,318.54	1,155,022.02	40.1	69.3	109.4	153,554.66	Georgia
Idaho	1,303.2	4,083,064.28	2,309,426.34	262.5	81.2	343.7	259,799.81	146,901.28	11.3	8.8	21.1	295,826.04	Idaho
Illinois	2,382.6	26,335,289.15	12,227,814.13	797.2	31.8	829.1	4,828,273.38	2,140,947.41	171.3	4.7	176.0	172,233.90	Illinois
Indiana	1,588.1	11,614,982.88	5,709,520.06	357.7		357.7	1,345,212.33	672,604.14	61.7		61.7	253,991.08	Indiana
Iowa	3,142.8	7,718,753.31	3,218,939.30	160.1	89.5	249.8	359.21	179.60				4,059.10	Iowa
Kansas	3,288.9	7,902,980.10	3,784,107.14	456.6	50.5	506.1	501,861.94	231,572.11	34.9	14.7	49.6	140,384.42	Kansas
Kentucky	1,576.5	9,378,644.10	2,792,008.17	273.8	125.8	399.4	423,506.25	204,025.56	35.1	1.8	36.9	65,279.39	Kentucky
Louisiana	1,466.4	7,154,869.98	3,373,317.89	189.0	18.9	208.8	751,852.47	307,329.64		.4	.4	135,581.56	Louisiana
Maine	623.5	4,080,022.00	1,645,510.88	88.2		88.2	1,171,868.57	485,461.29	15.0		15.0	336,834.18	Maine
Maryland	715.9	2,034,351.48	879,413.15	57.6	4.2	61.8	115,380.15	57,890.07	6.2		9.2	15,212.22	Maryland
Massachusetts	733.8	10,481,240.03	2,870,892.05	91.1		91.1	1,581,817.18	600,333.58	18.8		16.9	782,822.90	Massachusetts
Michigan	1,792.5	12,746,800.06	5,463,280.21	398.9	51.2	450.1	752,402.22	317,371.60	20.7	1.8	22.5	1,499,657.62	Michigan
Minnesota	3,954.0	8,371,819.88	3,861,797.71	75.4	317.7	393.1	247,848.39	73,029.32	4.5	1.2	5.7	14,252.31	Minnesota
Mississippi	1,765.3	4,398,821.97	2,160,957.43	184.6	87.8	272.5	242,383.87	119,750.39		12.8	12.9	3,693,892.99	Mississippi
Missouri	2,688.0	8,369,726.04	3,722,843.44	259.0	64.2	323.2	331,772.63	165,886.30	13.0		13.0	153,186.88	Missouri
Montana	2,008.7	10,350,196.87	5,809,841.29	747.8	137.8	885.4	1,516,250.48	864,573.90	66.1	51.0	117.1	1,389,878.52	Montana
Nebraska	3,863.5	10,529,886.19	4,920,147.04	278.1	224.0	502.1	1,318,421.27	642,490.66	66.0	51.0	117.0	393,839.99	Nebraska
Nevada	1,088.5	3,205,089.67	2,110,197.38	108.6	288.9	378.5	219,286.77	160,877.03		20.7	20.7	180,328.97	Nevada
New Hampshire	385.9	1,897,842.77	581,926.77	30.7	9.2	38.9						96,284.13	New Hampshire
New Jersey	571.2	4,988,514.39	1,711,603.78	58.7		58.7	1,419,234.56	652,846.01	7.6	.9	8.5	322,166.21	New Jersey
New Mexico	2,028.1	5,164,755.89	3,122,562.98	198.0	98.9	296.9	138,374.83	85,156.34	.3	1.7	2.0	120,762.84	New Mexico
New York	2,728.1	38,319,721.40	14,006,619.88	737.7	7.0	744.7	3,806,900.00	1,325,975.00	59.4		59.4	61,650.24	New York
North Carolina	2,027.9	5,659,111.65	2,735,593.12	171.8	38.8	208.9	318,797.14	151,373.14	25.7	8.0	33.7	1,625,359.86	North Carolina
North Dakota	4,547.0	4,121,489.14	2,101,066.56	426.5	388.2	813.7	726,895.44	382,269.37	52.1	182.7	234.8	796,377.77	North Dakota
Ohio	2,611.6	15,945,746.68	5,397,367.21	282.4	34.4	296.9	2,535,044.97	1,025,072.62	52.5	14.0	66.5	381,084.75	Ohio
Oklahoma	2,014.6	7,322,837.10	3,752,419.06	256.3	127.1	393.4	1,013,749.65	242,857.93	27.7	4.8	32.3	72,517.96	Oklahoma
Oregon	1,325.6	9,765,365.28	3,754,867.93	252.8	62.2	315.0	250,581.00	152,785.03	25.3		25.3	351,040.53	Oregon
Pennsylvania	2,683.9	13,877,971.48	8,113,974.43	352.5		352.5	888,060.04	312,897.80	18.2		18.2	516,420.00	Pennsylvania
Rhode Island	234.9	1,724,033.80	596,921.92	22.8		22.8	37,587.29	18,793.64	.1		.1	98,339.73	Rhode Island
South Carolina	1,890.0	4,510,632.52	2,062,898.52	78.2	116.3	194.5						255,046.71	South Carolina
South Dakota	3,876.5	5,204,653.80	2,720,360.50	265.0	223.5	488.5	452,606.14	195,527.91	4.1	84.2	88.3	60,633.20	South Dakota
Tennessee	1,600.7	1,872,218.13	956,800.30	63.8	8.5	73.1	380,641.70	190,320.84	26.0		26.0	1,827,439.03	Tennessee
Texas	7,101.8	18,930,576.83	9,197,564.88	763.8	307.9	1,071.7	4,251,779.67	1,805,300.78	198.6	62.3	260.9	1,855,537.30	Texas
Utah	1,052.1	2,048,388.38	1,123,194.11	108.5	80.2	188.7	496,494.08	365,581.64	23.1	13.7	36.8	306,642.55	Utah
Vermont	301.9	1,431,371.38	579,130.72	36.8	4.7	41.5						3,136.18	Vermont
Virginia	1,681.1	5,133,255.93	2,361,239.75	218.5	37.8	256.4	906,977.38	453,488.87	46.3		46.3	294,505.28	Virginia
Washington	984.7	8,239,912.08	2,748,272.97	178.4	43.7	222.1	502,213.33	270,500.00	18.6		18.8	627,729.64	Washington
West Virginia	786.4	5,355,066.35	2,222,847.98	132.4	25.4	157.8	578,382.90	276,019.26	18.0		18.0	53,090.85	West Virginia
Wisconsin	2,438.2	9,213,143.74	3,778,534.39	280.2	81.5	361.7	379,356.87	113,297.91	16.1	4.5	20.6	408.69	Wisconsin
Wyoming	1,683.2	4,484,065.69	2,639,186.89	336.4	221.4	556.8	140,754.29	81,093.75	19.7	21.9	41.8	136,098.22	Wyoming
Hawaii	47.8	1,505,296.22	614,835.17	43.2		43.2	160,999.13	62,208.07	1.7		1.7	1,506,748.30	Hawaii
TOTALS	90,636.8	378,478,076.34	179,924,029.38	11,884.3	4,206.5	16,090.8	41,879,777.20	18,584,139.86	1,373.3	650.7	2,024.0	27,124,941.80	TOTALS

[1]The term stage construction refers to additional work done on projects previously improved with Federal-aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 12, NO. 8 ▽ OCTOBER, 1931

A VIEW OF THE SUBGRADE LABORATORY AT ARLINGTON FARM, VA.

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BUREAU OF PUBLIC ROADS

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done the conclusions formulated must be considered as specifically pertinent only to the described conditions

VOL. 12, NO. 8 OCTOBER, 1931 G. P. St. CLAIR, Editor

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PROCEDURES FOR TESTING SOILS FOR THE DETERMINATION OF THE SUBGRADE SOIL CONSTANTS

By A. M. WINTERMYER, Assistant Highway Engineer, E. A. WILLIS, Assistant Highway Engineer and R. C. THOREEN, Junior Highway Engineer, United States Bureau of Public Roads

THIS is the fourth of a series of articles on the subject of subgrade soils. The reports published in the June and July, 1931, issues of PUBLIC ROADS discussed the soil test constants, their significance, and their application in practice. The report published in the September issue described the procedure for making subgrade soil surveys in the field. The purpose of the present report is to acquaint the reader with the procedure employed in testing soils in the subgrade laboratory of the bureau at Arlington, Va.

PREPARATION OF SAMPLE

1. *Apparatus.*—The apparatus consists of the following:

A balance sensitive to 0.1 gram.

A mortar and rubber-covered pestle suitable for breaking up the aggregations of soil particles.

A series of sieves, of square-mesh wire cloth, conforming to the requirements of the standard specifications for sieves for testing purposes of the American Society for Testing Materials (serial designation E–11). The sizes required are shown in Table 1.

A riffle sampler or sample splitter, for quartering the samples.

TABLE 1.—*Requirements for sieve openings and wire diameters with permissible variations*

Mesh designation, U. S. standard sieve series	Sieve opening		Wire diameter		Tolerance			
					Average opening	Wire diameter		Maximum opening
						Under	Over	
No.	*Millimeters*	*Inches*	*Millimeters*	*Inches*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
4	4.76	0.1870	1.27	0.050	±3	15	30	10
10	2.00	.0787	.76	.0299	±3	15	30	10
40	.42	.0165	.25	.0098	±5	15	30	25
200	.074	.0029	.053	.0021	±8	15	35	60

2. *Sample.*—The soil sample as received from the field is dried thoroughly in the air. A representative test sample of the amount required to perform the desired tests is then selected by the method of quartering or by the use of a sampler. The amounts of material required to perform the individual tests are as follows:

(*a*) For the mechanical analysis, material passing No. 10 sieve is required in amounts equal to 115 grams of sandy soils and 65 grams of either silt or clay soils.

(*b*) For the physical tests, material passing the No. 40 sieve is required in total amount equal to 200 grams, allocated as follows:

	Grams
Liquid limit	30
Plastic limit	15
Centrifuge moisture equivalent	10
Field moisture equivalent	50
Volumetric shrinkage	30
Flocculation and check tests	65

(*c*) Physical tests of binder material are performed only when it is desired to investigate the properties of the active constituents in sand-clay and gravel road-surfacing materials. In this case 100 grams of the material passing the No. 200 sieve are required and the tests are performed in the following order: Field moisture equivalent, plastic limit, liquid limit, volumetric shrinkage, and centrifuge moisture equivalent. The material remaining after the conclusion of each test is used in the next test, except that the test for the centrifuge moisture equivalent is made with material not used in previous tests.

FIGURE 1.—BUOYOUCOS HYDROMETER, CYLINDER, AND SPECIAL MILK-SHAKE MACHINE

3. *Procedure.*—That portion of the air-dried sample selected for test is weighed and the weight recorded as the weight of the total test sample uncorrected for hygroscopic moisture. The test sample is separated by sieving with a No. 10 sieve. That fraction retained on the No. 10 sieve is ground in a mortar with a rubber-covered pestle until the aggregations of soil particles are broken up into the separate grains. The ground soil is then separated into two fractions by sieving with a No. 10 sieve.

That fraction retained after the second sieving is washed free of all fine material, dried, and weighed. This weight is recorded as the weight of coarse material. The coarse material after being washed and dried is sieved on the No. 4 sieve and the weight retained on the No. 4 sieve is recorded.

The fractions passing the No. 10 sieve in both sieving operations are thoroughly mixed together, and by the method of quartering or the use of a sampler a portion weighing approximately 115 grams for sandy soils and approximately 65 grams for silt and clay soils is selected for mechanical analysis.

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The remaining portion of the material passing the No. 10 sieve is then separated into two parts by means of a No. 40 sieve. The fraction retained on the No. 40 sieve is discarded. The fraction passing the No. 40 sieve is used for the physical tests.

MECHANICAL ANALYSIS BY COMBINED SIEVE AND HYDROMETER METHOD

1. *Apparatus.*—The apparatus consists of the following:

An analytical balance sensitive to 0.001 gram.

A special milk-shake machine with specially designed dispersion cup.

A hydrometer graduated to read grams of soil per liter of suspension. The milk-shake machine and hydrometer, illustrated in Figure 1, were designed by G. J. Bouyoucos and are described in Soil Science, (vol. 23, No. 4, April, 1927, pp. 319 to 330).

A glass graduate 18 inches high and 2½ inches in diameter and graduated for a volume of 1,000 cubic centimeters.

A Fahrenheit thermometer accurate to 1°.

A series of sieves, of square-mesh wire cloth, conforming to the requirements of the standard specifications for sieves for testing purposes of the American Society for Testing Materials (serial designation E–11). The sieves required are shown in Table 2.

A water bath for maintaining the soil suspension at a constant temperature during the hydrometer analysis. This is an insulated zinc tank and maintains the temperature of the suspension at faucet-water temperature. It is illustrated in Figure 2.

A glass cylinder 9½ inches high and 2 inches in diameter, having a capacity of about 425 cubic centimeters.

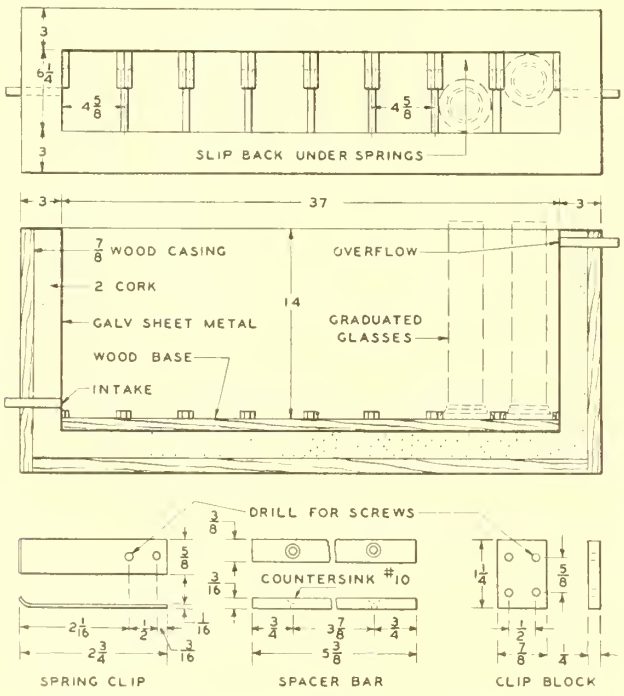

FIGURE 2.—TANK FOR GRADUATED GLASSES, USED FOR MAINTAINING SOIL SUSPENSIONS AT CONSTANT TEMPERATURE DURING HYDROMETER ANALYSIS

2. *Sample.*—Of that portion of the total sample selected for the mechanical analysis, 15 grams are used to determine the hygroscopic moisture and the remainder is used for the combined sieve and hydrometer analysis.

3. *Determination of hygroscopic moisture.*—The 15 grams selected for this purpose are dried to constant weight in an oven at 110° C., weighed, and the results recorded.

4. *Flocculation test.*—Five cubic centimeters of soil particles (weight in grams equal to five times the specific gravity of the soil particles) of that fraction passing the No. 40 sieve are placed in a graduate and

TABLE 2.—*Requirements for sieve openings and wire diameters with permissible variations*

Mesh designation, U. S. standard sieve series	Sieve opening		Wire diameter		Tolerance			
					Average opening	Wire diameter		Maximum opening
						Under	Over	
	Millimeters	*Inches*	*Millimeters*	*Inches*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
No.								
20	0.84	0.0331	0.42	0.0165	±5	15	30	25
40	.42	.0165	.25	.0098	±5	15	30	25
60	.25	.0098	.162	.0064	±6	15	35	40
140	.105	.0041	.074	.0029	±8	15	35	60
200	.074	.0029	.053	.0021	±8	15	35	60

vigorously shaken with 45 cubic centimeters of distilled water for two minutes. The graduate is then set aside for a period of 24 hours. If during this period there is evidence of flocculation, the fact is recorded, together with a note regarding the extent of flocculation and the approximate time at which it became evident.

5. *Hydrometer test procedure.*—The portion of air-dried soil selected for mechanical analysis is dispersed by one of the three methods described below. The method to be used is determined by the plasticity index of the soil.

A. In the case of soils having a plasticity index between 0 and 5, the soil is placed in the special dispersion cup and distilled water is added until the cup is within 2 inches of being full. A deflocculating agent, 20 cubic centimeters of sodium silicate solution (3° Baumé at 76° F.), is then added and the contents of the cup are mixed by the special milk-shake machine for a period of five minutes.

B. In the case of soils having a plasticity index between 5 and 20 the soil is placed in a small evaporating dish and completely covered with water. It is allowed to soften under water for a period of at least 18 hours. After the soil has softened it is washed into the special dispersion cup and dispersed in the same manner as in method *A*, except that the time of dispersion is increased to 10 minutes.

C. In the case of soils having a plasticity index greater than 20 the soil is placed in a glass cylinder and to this is added 100 cubic centimeters of 6 per cent hydrogen peroxide. The cylinder is shaken until the soil is completely wetted. The cylinder is then covered with a watch glass and placed in an oven at a temperature of 110° C. for 1 hour, after which it is removed from the oven and allowed to stand for at least 18 hours. The peroxide is used to assist in the dispersion rather than to remove the organic matter. After the soil has been treated with peroxide as described above it is washed into the special dispersion cup and dispersed in the same manner as in method *A*, except that the time of dispersion is increased to 15 minutes.

It is important in all cases to see that the paddle on the dispersion machine is replaced as soon as it shows signs of wear.

After dispersion the mixture is transferred to the glass graduate, and distilled water having the same temperature as the constant temperature bath is added until the mixture attains a volume of 1,000 cubic centimeters. The graduate containing the soil suspension is then placed in the constant temperature bath. The suspension is stirred frequently with a glass rod to prevent settlement of particles in suspension. When the soil suspension attains the temperature of the bath

the graduate is removed and its contents thoroughly shaken for one minute, the palm of the hand being used as a stopper over the mouth of the graduate. At the conclusion of this shaking the time is recorded, the graduate placed in the bath, and readings taken with the hydrometer at the end of both one and two minutes. The hydrometer is read at the top of the meniscus formed by the suspension around its stem to the nearest one-half gram per liter. Subsequent readings are taken at intervals of 5, 15, 30, 60, 250, and 1,440 minutes after the beginning of sedimentation. Readings on the thermometer placed in the constant temperature bath are made coincidentally with the hydrometer readings and recorded.

After each reading except the 1-minute reading, the hydrometer is very carefully removed from the soil suspension in such a manner as to cause no disturbance in the suspension, wiped clean, and laid aside. Fifteen or twenty seconds before the time for a reading it is again slowly and carefully placed in the soil suspension. This operation prevents soil particles from accumulating on the hydrometer and also prevents the hydrometer from reducing the horizontal sectional area of the suspension through which the soil particles settle. The reading is not taken until the hydrometer has come to rest.

6. *Sieve analysis.*—At the conclusion of the final reading the suspension is washed on a No. 200 sieve. That fraction retained on the No. 200 sieve is dried and then analyzed in a nest of sieves consisting of one each of the following: Nos. 20, 40, 60, and 140.

COMPUTATION OF DATA GIVEN BY MECHANICAL ANALYSIS

The data obtained from the three parts of the test, i. e. the separation of coarse material in the preparation of the sample, the hydrometer analysis, and the sieve analysis—are computed and combined as described below and illustrated in Tables 3 and 4.

1. *Hygroscopic moisture.*—The hygroscopic moisture is expressed as a percentage of the weight of the oven-dried soil and is one hundred times the quantity obtained by dividing the difference between the weight of the air-dried and the weight of the oven-dried soil by the weight of the oven-dried soil. The method of computation is illustrated in Table 3.

To correct the weight of the air-dried sample for hygroscopic moisture the given value is multiplied by the expression

$$\frac{100}{100 + \text{per cent of hygroscopic moisture}}$$

This factor, as illustrated in Table 3, equals

$$\frac{100}{100 + 2.53} = 0.975$$

Thus in Table 4 the weight of air-dried sample, 99.0 grams, multiplied by 0.975 equals 96.5 grams, the weight of dry soil dispersed.

2. *Coarse material.*—From the weight of the air-dried total test sample (318.3 grams, Table 3) the weight of the oven-dried fraction retained on the No. 10 sieve (56.2 grams, Table 3) is subtracted. The difference (262.1 grams, Table 3) is assumed to equal the weight of the air-dried fraction passing the No. 10 sieve. According to this assumption, no hygroscopic moisture is contained in the air-dried particles retained on the No. 10 sieve, when as a matter of fact a small

TABLE 3.—*Hygroscopic moisture and coarse material determinations for sample 4,422X*

HYGROSCOPIC MOISTURE	
Weight of air-dried soil, grams	15.00
Weight of dish and air-dried soil, grams	41.37
Weight of dish and oven-dried soil, grams	41.00
Loss in weight, grams	0.37
Weight of oven-dried soil, grams	14.63
Hygroscopic moisture, percentage of weight of oven-dried soil	2.53
Hygroscopic moisture correction factor	0.975
COARSE MATERIAL	
Weight of total test sample, air-dried, grams	318.3
Weight of washed and oven-dried fraction retained on No. 10 sieve, grams	56.2
Weight of fraction passing No. 10 sieve, air-dried, grams	262.1
Weight of fraction passing No. 10 sieve corrected for hygroscopic moisture, grams	255.5
Weight of total test sample corrected for hygroscopic moisture, grams	311.7
Weight of fraction retained on No. 4 sieve, oven-dried, grams	40.6
Fraction retained on No. 4 sieve, percentage of corrected weight of total test sample	13.0
Fraction retained on No. 10 sieve, percentage of corrected weight of total test sample	18.0

percentage of moisture may be present in this fraction. This amount of moisture, compared with that held in the pores of the fraction passing the No. 10 sieve, is relatively small. Therefore any error produced by the assumption as stated is considered negligible in amount. The weight of the fraction passing the No. 10 sieve is corrected for hygroscopic moisture (255.5 grams, Table 3). To this value is added the weight of the oven-dried fraction retained on the No. 10 sieve to obtain the weight of the total test sample corrected for hygroscopic moisture (311.7 grams, Table 3). The fractions retained on both the No. 4 and the No. 10 sieve are expressed as percentages of the corrected weight of the total test sample (13.0 per cent and 18.0 per cent, respectively, Table 3).

3. *Percentage of soil in suspension.*—For temperatures of the constant temperature bath other than that at which the hydrometer was calibrated, the hydrometer readings are corrected in accordance with temperature correction factors such as are shown graphically as ΔR in Figure 3, A. A temperature correction curve of this type should be determined experimentally for each hydrometer in use. Thus in Table 4 the first hydrometer reading, 34, taken at 70° F. becomes 34.4 when corrected for temperature in accordance with Figure 3, A.

The percentage of the dispersed soil in suspension represented by different corrected hydrometer readings depends upon both the amount and the specific gravity of the soil dispersed.

If the specific gravity of the soil is 2.65, the hydrometer reading gives the weight of soil remaining in suspension in grams per liter of the mixture or suspension. The percentage of dispersed soil remaining in suspension is given by the expression

$$P = \frac{R}{W} \times 100$$

where P = percentage of originally dispersed soil remaining in suspension.
R = hydrometer reading.
W = weight of soil originally dispersed, in grams per liter of suspension.

If, as is the customary procedure, the volume of the suspension is 1 liter, the term W may be taken as the

total weight of soil originally dispersed, without the qualifying phrase, "per liter of suspension."

If the specific gravity of the soil is other than 2.65, the percentage of originally dispersed soil remaining in suspension is given by the expression

$$P = \frac{Ra}{W} \times 100$$

in which a is a constant depending on the density of the suspension. The value of a, for a specific gravity G and a water density at 67° F. of 0.9984, is given by the equation

$$a = \frac{2.6500 - 0.9984}{2.6500} \times \frac{G}{G - 0.9984}$$

Following are values of a for different values of the specific gravity:

Specific gravity, G	Constant, a
2.95	0.94
2.85	0.96
2.75	0.98
2.65	1.00
2.55	1.02
2.45	1.05
2.35	1.08

The percentage of the dispersed soil remaining in suspension may be obtained from this table by interpolation. It is sufficiently accurate, however, to select the constant for the specific gravity closest to that of the particular soil tested. Thus, in Table 4, sample No. 4,422X has a specific gravity of 2.41, and consequently the constant, 1.05, corresponding to a specific gravity of 2.45, is used.

A corrected hydrometer reading of 34.4 in Table 4, therefore, indicates a percentage of dispersed soil in suspension,

$$P = \frac{34.4 \times 1.05}{96.5 \times .01} = 37.4 \text{ per cent}$$

For any hydrometer reading R the percentage of dispersed soil in suspension $= R \times 1.088$.

The percentage of the total test sample, including the fraction retained on the No. 10 sieve, is obtained by multiplying this result by the expression

$$\frac{100 - \text{per cent retained on the No. 10 sieve}}{100}$$

Thus in Table 4, for a hydrometer reading of 34.4, the percentage of the total sample remaining in suspension is obtained by the computation

$$P_1 = 37.4 \times \frac{100 - 18.1}{100} = 30.7$$

This computation can be combined with the one above, allowing the percentage of the total sample remaining in suspension to be computed from the corrected hydrometer reading R. We have, therefore,

$$P_1 = R \times 1.088 \times 0.82 = 0.892\ R$$

Thus for a corrected hydrometer reading of 25.9, Table 4, we obtain the percentage of the total sample in suspension,

$$P_1 = 25.9 \times 0.892 = 23.1$$

TABLE 4.—*Sieve and hydrometer analysis for sample No. 4,422X*

Percentage of sample retained on No. 10 sieve	18.0
Weight of air-dried sample, grams	99.0
Weight of dry soil dispersed, grams	96.5
Weight of total test sample represented by weight of dry soil, grams	117.7
Specific gravity	2.41
Plasticity index	8.0
Flocculation	None.

DETERMINATION OF PERCENTAGE OF SOIL IN SUSPENSION

Date tested	Time observed	Temperature	Hydrometer reading: Original	Hydrometer reading: Corrected for temperature (fig. 3, A).	Percentage of dispersed sample remaining in suspension, P	Percentage of total test sample remaining in suspension, P_1
		°F.				
July 29, 1929	9.41 a. m.					
Do	9.42 a. m.	70	34.0	34.4	37.4	30.7
Do	9.43 a. m.	70	25.5	25.9	28.2	23.1
Do	9.46 a. m.	70	19.0	19.4	21.1	17.3
Do	9.56 a. m.	70	15.0	15.4	16.8	13.7
Do	10.11 a. m.	70	12.0	12.4	13.5	11.1
Do	10.41 a. m.	70	10.5	10.9	11.9	9.7
Do	1.51 p. m.	70	7.0	7.4	8.1	6.6
July 30, 1929	9.41 a. m.	68	3.0	3.1	3.4	2.8

DETERMINATION OF SIZE OF SOIL PARTICLES IN SUSPENSION

Original hydrometer reading	Period of sedimentation, T	Grain diameter[1] D	Temperature	Correction coefficients: K_L, Fig. 3, B	Correction coefficients: K_G, Fig. 3, C	Correction coefficients: K_a, Fig. 3, D	Corrected grain diameter
	Minutes	*Millimeters*	°F.				*Millimeters*
34.0	1	0.078	70	0.48	1.08	0.98	0.0396
25.5	2	.055	70	.50	1.08	.98	.0291
19.0	5	.035	70	.51	1.08	.98	.0189
15.0	15	.020	70	.52	1.08	.98	.0110
12.0	30	.014	70	.53	1.08	.98	.0079
10.5	60	.010	70	.53	1.08	.98	.0056
7.0	250	.005	70	.54	1.08	.98	.0029
3.0	1,440	.002	68	.55	1.08	.99	.0012

[1] See table 5.

SIEVE ANALYSIS

Fraction	Weight	Percentage of total test sample
	Grams	
Passing No. 10, retained on No. 20	2.35	2.0
Passing No. 20, retained on No. 40	2.59	2.2
Passing No. 40, retained on No. 60	4.12	3.5
Passing No. 60, retained on No. 140	9.41	8.0
Passing No. 140, retained on No. 200	12.11	10.3

4. *Diameter of soil particles in suspension.*—The maximum diameters of the particles in suspension, based on Stokes's law for assumed conditions suggested in part by G. J. Bouyoucos, are shown in Table 5. (See Soil Science, vol. 26, No. 3, September, 1928, p. 234.)

According to Stokes's law,

$$d = \sqrt{\frac{30nL}{980\ (G - G_1)\ T}}$$

In this equation—

d = maximum grain diameter in millimeters.

n = coefficient of viscosity of the suspending medium (in this case water) in poises. Varies with changes in temperature of the suspending medium.

L = distance in centimeters through which soil particles settle in a given period of time.

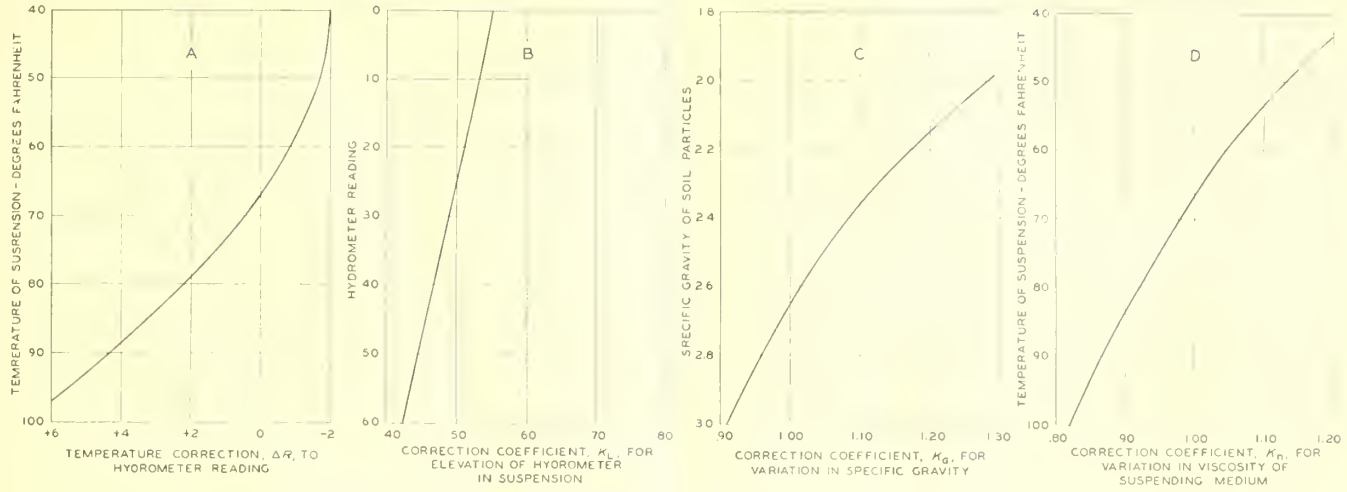
A
TEMPERATURE OF SUSPENSION - DEGREES FAHRENHEIT
TEMPERATURE CORRECTION, ΔR, TO HYDROMETER READING
B
HYDROMETER READING
CORRECTION COEFFICIENT, K_L, FOR ELEVATION OF HYDROMETER IN SUSPENSION
C
SPECIFIC GRAVITY OF SOIL PARTICLES
CORRECTION COEFFICIENT, K_G, FOR VARIATION IN SPECIFIC GRAVITY
D
TEMPERATURE OF SUSPENSION - DEGREES FAHRENHEIT
CORRECTION COEFFICIENT, K_n, FOR VARIATION IN VISCOSITY OF SUSPENDING MEDIUM

FIGURE 3.—CORRECTION CURVES FOR USE IN HYDROMETER ANALYSIS

T = time in minutes, period of sedimentation.
G = specific gravity of soil particles.
G_1 = specific gravity of the suspending medium. In this case $G_1 = 0.9984$, or approximately 1.0.

TABLE 5.—*Maximum grain diameters in suspension under assumed conditions*

Time	Maximum grain diameter in suspension
Minutes	*Millimeters*
1	0.078
2	.055
5	.035
15	.020
30	.014
60	.010
250	.005
1,440	.002

The grain diameters given in Table 5 are computed according to the following assumptions:

L, the distance through which the particle falls, is constant and equal to 32.5 centimeters.

n, the coefficient of viscosity, equals 0.0102, that of water at 67° F.[1]

G, the specific gravity of the soil particles, is constant and equal to 2.65.

As a matter of fact, the hydrometer reading is dependent, not on particles distributed throughout a depth of 32.5 centimeters in the suspension, but on those existing in that portion of the suspension holding the hydrometer.

In order to use Stokes's law to determine the diameter of the particles it is necessary to know the distance through which these particles fall in a given time. Since the density throughout a suspension is not uniform and varies with the grading of the material in suspension and the time of sedimentation, a fixed distance can not be used. For hydrometers of certain shapes the depth of the center of volume of the hydrometer below the surface of the suspension could be taken as the distance through which the particles may be assumed to fall. In the case of the Bouyoucos hydrometer it has been found by experiment that for the methods of dispersion described in this procedure an assumed distance which bears a constant ratio to the depth of the hydrometer in the suspension, but which is less than the distance indicated by the center of volume of the hydrometer, gives closer agreement to mechanical analysis performed by the pipette methods. The assumed distance of fall has been taken as 0.42 of the distance from the surface of the suspension to the elevation of the bottom of the hydrometer.

The specific gravities of the soil particles and the temperature of the suspension are likely to vary from those assumed in the preparation of Table 3. A better approximation to the true diameters of the soil particles is obtained by applying correction coefficients to the values given by Table 5.

Curves from which these coefficients may be derived are given in Figure 3. The correction coefficients for elevation of hydrometer (fig. 3, B) are obtained experimentally for each hydrometer in use. The coefficients for the specific gravity and the viscosity correction (figs. 3, C and 3, D, respectively) are independent of the apparatus used in the test.

Multiplication by the coefficient shown in Figure 3, B gives the maximum grain size at the reference elevation in the suspension instead of that at a depth of 32.5 centimeters. This coefficient varies with the hydrometer reading and is given by the expression,

$$K_L = \sqrt{\frac{\text{assumed depth of fall in centimeters}}{32.5}}$$

Multiplication by the coefficient shown in Figure 3, C corrects for variation in specific gravity[2] from that on which the sizes given in Table 5 are based and is given by the expression

$$K_G = \sqrt{\frac{1.65}{\text{specific gravity of soil particle} - 1}}$$

Multiplication by the coefficient shown in Figure 3, D corrects for the viscosity of water at temperatures other than 67° F., the temperature of the suspension assumed in the preparation of Table 5. The viscosity correction coefficient is given by the expression

$$K_n = \sqrt{\frac{\text{viscosity coefficient at given temperature}}{0.0102}}$$

The application of these coefficients is illustrated in Table 4. After a period of sedimentation of one minute

[1] Smithsonian Physical Tables, seventh revised edition, 1921, p. 155.

[2] The specific gravity of the soil should be obtained by the pycnometer method

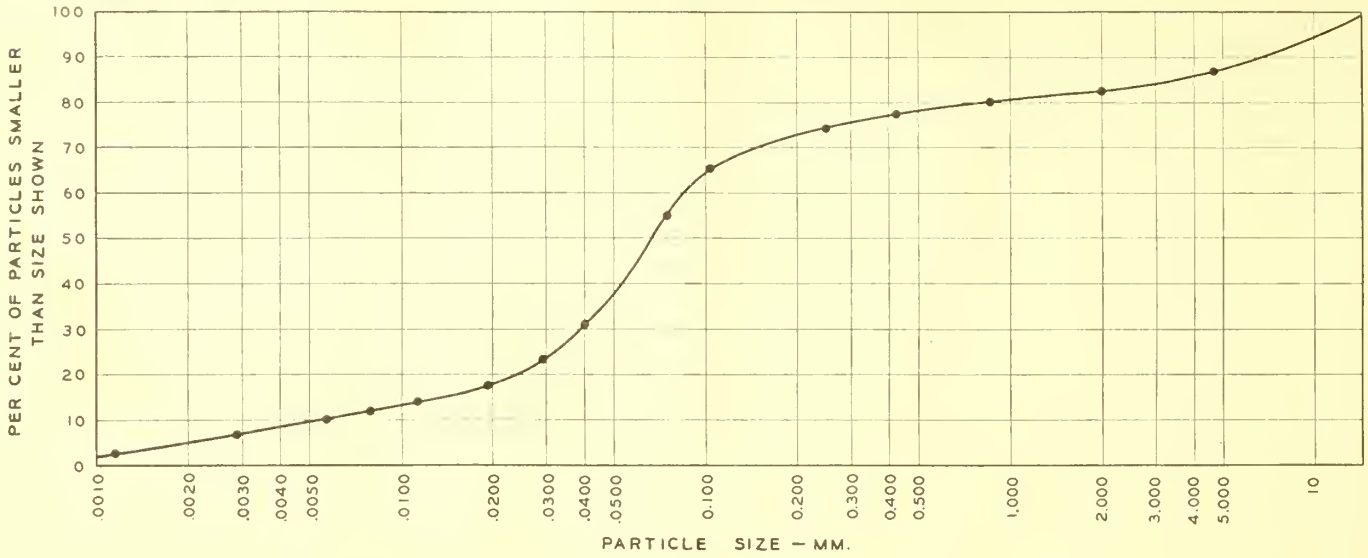

FIGURE 4.—GRAIN-SIZE ACCUMULATION CURVE FOR SOIL SAMPLE 4,422X

the grain diameter indicated by Table 5 is 0.078 millimeters. The uncorrected hydrometer reading was 34, the specific gravity was 2.41, and the temperature of the suspension was 70° F. The correction coefficients in this case are 0.48, corresponding to a hydrometer reading of 34 (fig. 3, B); 1.08, corresponding to a specific gravity of 2.41 (fig. 3, C); and 0.98, corresponding to a water temperature of 70° F. (fig. 3, D). The corrected grain diameter then becomes

$$0.078 \text{ millimeter} \times 0.48 \times 1.08 \times 0.98 = 0.040 \text{ millimeter}$$

5. *Sieve analysis.*—The percentage of the soil sample retained on each of the sieves in the sieve analysis is obtained by dividing the weight of fraction retained on each sieve by the weight of the oven-dried fraction dispersed (117.7 grams, Table 4) and multiplying by 100.

6. *Plotting.*—The percentages of grains of different diameters are plotted to a logarithmic scale to obtain "soil grain-diameter accumulation curves." Figure 4 shows a curve of this character representing the mechanical analysis of soil sample No. 4,422X.

7. *Record.*—The results are reported as follows:

Particles larger than 2 millimeters, per cent.
Coarse sand, 2.0 millimeters to 0.25 millimeter, per cent.
Fine sand, 0.25 millimeter to 0.05 millimeter, per cent.
Silt, 0.05 millimeter to 0.005 millimeter, per cent.
Clay, smaller than 0.005 millimeter, per cent.
Colloids, smaller than 0.001 millimeter, per cent.

DETERMINATION OF LIQUID LIMIT

1. *Definition.*—The liquid limit of a soil is that moisture content, expressed as a percentage of the weight of the oven-dried soil, at which the soil will just begin to flow when lightly jarred ten times.

2. *Apparatus.*—The apparatus consists of the following:

A porcelain evaporating dish about 4½ inches in diameter.
A flexible spatula having a blade about 3 inches long and about ¾ inch wide.
A grooving tool of dimensions shown in Figure 5.
Matched watch glasses which are held together by a suitable clamp and fit sufficiently tight to prevent loss of moisture during weighing.
An analytical balance sensitive to 0.001 gram.

3. *Sample.*—A sample weighing about 30 grams is taken from the thoroughly mixed portion of the material passing the No. 40 sieve.

4. *Procedure.*—The air-dried soil is placed in the evaporating dish and thoroughly mixed with water until the mass becomes pasty. The mass of soil is then shaped into a smooth layer about three-eighths inch thick at the center and divided into two portions with the grooving tool, as shown in Figure 6, top.

The dish is held firmly in one hand, with the groove parallel to the line of sight, and tapped lightly with a

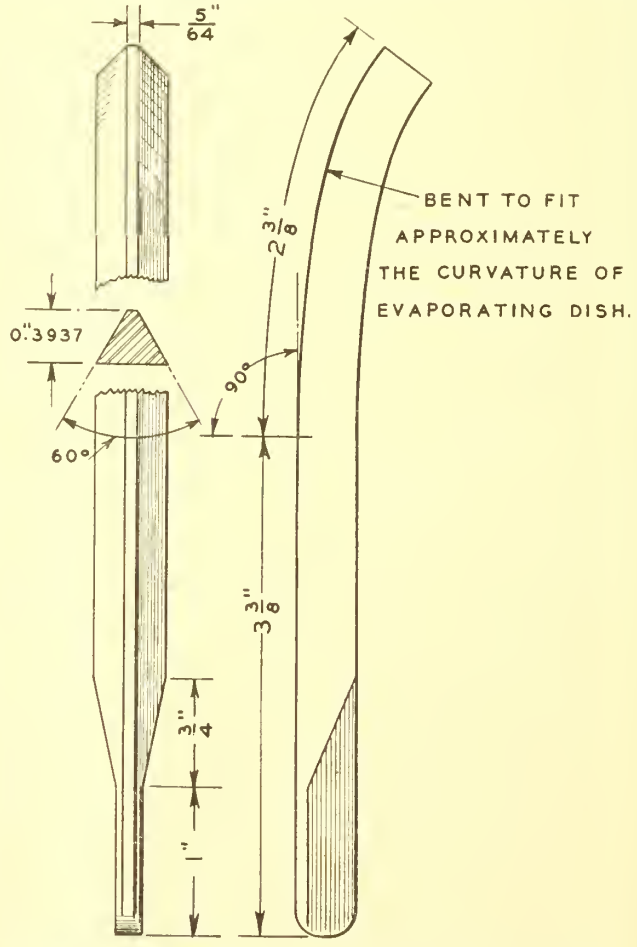

FIGURE 5.—GROOVING TOOL USED IN LIQUID LIMIT TEST

FIGURE 6.—DIAGRAM ILLUSTRATING LIQUID LIMIT TEST

horizontal motion against the palm of the other hand ten times. If the lower edges of the two soil portions do not flow together as shown in Figure 6, bottom, after ten blows have been struck, the moisture content is below the liquid limit. More water should be added and the procedure repeated. If the lower edges meet before ten blows have been struck, the moisture content is above the liquid limit, and dry soil should be added and the procedure repeated.

When the lower edges of the two portions of the soil cake just flow together as shown in Figure 6, bottom, after ten blows have been struck, the moisture content equals the liquid limit. To determine definitely whether the two portions are actually joined, the spatula is used to push one away from the other. If the two portions separate along the original line of division, the end point has not been reached, and the procedure is repeated with the addition of a small amount of water.

A small quantity of soil from that portion of the soil cake which has flowed is removed and placed in a pair of watch glasses. The watch glasses and soil are then weighed and the weight recorded as the weight of glass and wet soil (37.49 grams in Table 6). The soil in the glasses is oven-dried to constant weight at a temperature of 110° C. and weighed. This weight is recorded as the weight of glass and dry soil (28.15 grams, Table 6). The loss in weight due to drying (37.49 grams − 28.15 grams = 9.34 grams, Table 6) is recorded as the weight of water.

5. *Calculation.*—The liquid limit is expressed as the moisture content in percentage of the weight of the oven-dried soil. It is computed from the following formula:

$$L.\,L. = \frac{\text{weight of water}}{\text{weight of dry soil}} \times 100$$

Thus in Table 6 the liquid limit of sample S 5,214 is

$$\frac{9.34}{15.06} \times 100 = 62.0 \text{ per cent}$$

DETERMINATION OF PLASTIC LIMIT

1. *Definition.*—The plastic limit of a soil is the lowest moisture content, expressed as a percentage of the weight of the oven-dried soil, at which the soil can be rolled into threads one-eighth inch in diameter without the threads breaking into pieces.

TABLE 6.—*Plasticity determinations for sample S 5,214*

LIQUID LIMIT TEST

Weight of glass and wet soil, grams	37. 49
Weight of glass and dry soil, grams	28. 15
Weight of glass, grams	13. 09
Weight of water, grams	9. 34
Weight of dry soil, grams	15. 06
Liquid limit, per cent	62. 0

PLASTIC LIMIT TEST

Weight of glass and wet soil, grams	32. 20
Weight of glass and dry soil, grams	28. 59
Weight of glass, grams	12. 17
Weight of water, grams	3. 61
Weight of dry soil, grams	16. 42
Plastic limit, per cent	22. 0
Plasticity index, per cent	40. 0

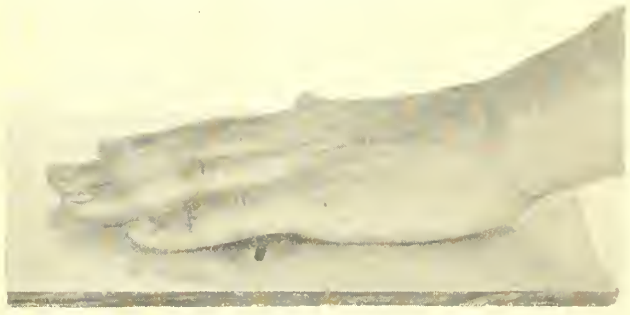

FIGURE 7.—ROLLING OF SOIL THREADS IN PLASTIC LIMIT TEST

2. *Apparatus.*—The apparatus consists of the following:

A porcelain evaporating dish about 4½ inches in diameter.

A flexible spatula having a blade about 3 inches long and about three-fourths inch wide.

A glass plate or piece of glazed paper on which to roll the sample.

Matched watch glasses which are held together by a suitable clamp and fit sufficiently tight to prevent loss of moisture during weighing.

An analytical balance sensitive to 0.001 gram.

3. *Sample.*—A sample weighing about 15 grams is taken from the thoroughly mixed portion of the material passing the No. 40 sieve.

FIGURE 8.—CRUMBLED SOIL THREADS RESULTING FROM PLASTIC LIMIT TEST

4. *Procedure.*—The air-dried soil is placed in the evaporating dish and mixed with water until the mass becomes plastic enough to be easily shaped into a ball. The ball of soil is then rolled between the palm of the hand and the glass plate or piece of glazed paper with just sufficient pressure to form the soil mass into a thread. (Fig. 7.) When the diameter of the resulting thread becomes one-eighth of an inch the soil is kneaded together and again rolled out. This process is continued until the crumbling of the soil (as shown in fig. 8) prevents the formation of the thread. The portions of

the crumbled soil are then gathered together and placed in watch glasses. The watch glasses and soil are weighed and the weight recorded as the weight of glass and wet soil (32.20 grams, Table 6). The soil in the glasses is then oven-dried to constant weight at a temperature of 110° C. and weighed. This weight is recorded as the weight of glass and dry soil (28.59 grams, Table 6). The loss in weight (32.20 grams − 28.59 grams = 3.61 grams, Table 6) is recorded as the weight of water.

5. *Calculations.*—The plastic limit is expressed as the moisture content in percentage of the weight of the oven-dry soil. It is computed from the following formula:

$$P.\ L. = \frac{\text{weight of water}}{\text{weight of dry soil}} \times 100$$

Thus in Table 6 the plastic limit of sample S 5,214 equals

$$\frac{3.61}{16.42} \times 100 = 22.0 \text{ per cent}$$

DETERMINATION OF PLASTICITY INDEX

1. *Definition.*—The plasticity index of a soil is the difference between its liquid limit and its plastic limit.

2. *Calculation.*—The plasticity index is calculated by the formula $P.\ I. = L.\ L. - P.\ L.$

Thus in Table 6 the plasticity index of sample S 5,214 equals 62.0 − 22.0 = 40.0 per cent.

DETERMINATION OF CENTRIFUGE MOISTURE EQUIVALENT

1. *Definition.*—The centrifuge moisture equivalent of a soil is the amount of moisture, expressed as a percentage of the weight of the oven-dried soil, retained by a soil which has been first saturated with water and then subjected to a force equal to one thousand times the force of gravity for one hour.

2. *Apparatus.*—The apparatus consists of the following:

A porcelain Gooch crucible with perforated bottom. The crucible is about 1½ inches in height and about 1 inch in diameter at the top and three-fourths of an inch at the bottom, outside dimensions.

A circular piece of filter paper just large enough to cover the inside bottom of the Gooch crucible.

A Babcock trunnion cup fitted with a brass cap and with a rubber stopper with a hole in the center, as shown in Figure 9.

A centrifuge of such size and so driven that a force equal to one thousand times the force of gravity may be exerted on the center of gravity of the soil sample.

An analytical balance sensitive to 0.001 gram.

3. *Sample.*—A 5-gram sample is taken from the thoroughly mixed portion of the material passing the No. 40 sieve.

4. *Number of tests.*—Tests are made in duplicate. Table 7 shows the record for the two tests of sample S 5,214.

5. *Procedure.*—The sample is placed in the Gooch crucible, in which has previously been placed a piece of wet filter paper which just covers the bottom of the crucible. The crucible is placed in a pan of water and the sample allowed to take up moisture until completely saturated, as indicated by the presence of free water on the surface of the sample. It is then placed in a humidifier for at least 12 hours to insure uniform distribution of moisture throughout the soil mass. All free water then remaining on the surface of the sample is poured off, and the crucible is placed in a Babcock cup fitted with a rubber stopper, as shown in Figure 10. The hole in the stopper should be large enough to receive the water forced from the soil by the centrifuging operation. In addition to receiving the ejected water the stopper serves also as a cushion for the crucible. The sample is centrifuged for a period of one hour at a speed which, for the diameter of head used, will exert a centrifugal force one thousand times the force of gravity upon the center of gravity of the soil sample. Immediately after centrifuging, the crucible and contents are weighed and the weight recorded as the weight of crucible and contents after centrifuging (15.74 grams, Table 7). The sample is then oven-dried to constant weight at a temperature of 110° C. and weighed. This weight is recorded as the weight of crucible and contents after drying (12.82 grams, Table 7).

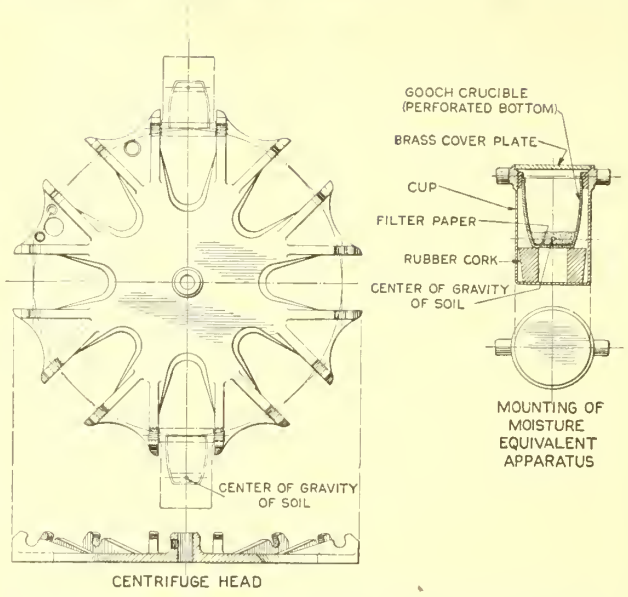

FIGURE 9.—APPARATUS FOR DETERMINING THE CENTRIFUGE MOISTURE EQUIVALENT

6. *Water-logging.*—When free water is observed on the top of the sample after the centrifuging operation the soil is said to have water-logged. This water is not removed, but is weighed with the sample.

7. *Calculation.*—The centrifuge moisture equivalent of the soil is calculated from the formula,

$$C.\ M.\ E. = \frac{(A - b) - (A_1 - b_1)}{A_1 - (c + b_1)} \times 100$$

in which

A = weight of crucible and contents after centrifuging.
A_1 = weight of crucible and contents after drying.
c = weight of crucible.
b = weight of filter paper wet.
b_1 = weight of filter paper dry.

Thus in Table 7 the centrifuge moisture equivalent of sample S 5,214, as given by the first test, is obtained by the computation,

$$C.\ M.\ E. = \frac{(15.74 - 0.20) - (12.82 - 0.10)}{12.82 - (7.86 + 0.10)} \times 100$$
$$= \frac{2.82}{4.86} \times 100 = 58.0 \text{ per cent.}$$

8. *Variation.*—The variation between the two values obtained should not exceed 1 per cent for values of the moisture equivalent up to 15 and 2 per cent for values above 15.

TABLE 7.—*Determination of centrifuge moisture equivalent for sample S 5,214*

Test No.	Weight [1] of—			Centrifuge moisture equivalent
	Crucible and contents after centrifuging	Crucible and contents after drying	Crucible	
	Grams	*Grams*	*Grams*	*Per cent*
1	15.74	12.82	7.86	[2] 58.0
2	16.03	13.00	8.02	[2] 60.0
Average				59.0

[1] Weight of filter paper: Wet, b=0.20 gram; dry, b_1=0.10 gram.
[2] Water-logged.

DETERMINATION OF FIELD MOISTURE EQUIVALENT

1. *Definition.*—The field moisture equivalent of a soil is defined as the minimum moisture content, expressed as a percentage of the weight of the oven-dried soil, at which a drop of water placed on a smoothed surface of the soil will not immediately be absorbed by the soil but will spread out over the surface and give it a shiny appearance.

2. *Apparatus.*—The apparatus consists of the following:

A porcelain evaporating dish about 4½ inches in diameter.
A flexible spatula having a blade about 3 inches long and about three-fourths inch wide.
A pipette, burette, or similar device for adding water dropwise.
Matched watch glasses, held together by a suitable clamp and fitting sufficiently tight to prevent loss of moisture during weighing.
An analytical balance sensitive to 0.001 gram.

3. *Sample.*—A sample weighing about 50 grams is taken from the thoroughly mixed portion of the material passing the No. 40 sieve.

4. *Procedure.*—The air-dried sample is placed in the evaporating dish and mixed with water. Water is added in small amounts and the sample is thoroughly mixed after each addition of water. When the wetted soil forms into balls under manipulation the sample is smoothed off with a light stroke of the spatula and a drop of water is placed on the smoothed surface. If the water immediately disappears a few more drops of water are added, and the procedure is repeated until the water does not immediately disappear but spreads over the smoothed surface and leaves a shiny appearance. A small portion of the soil on which the last drop was placed is then removed and placed between two watch glasses. The weight of the watch glasses and wet soil is determined and recorded (32.08 grams, Table 8). The sample is then oven-dried to constant weight at a temperature of 110° C. and weighed. This weight is recorded as the weight of glass and dry soil (26.29 grams, Table 8). The difference in weight (32.08 grams − 26.29 grams = 5.79 grams, Table 8) is recorded as the weight of water.

5. *Calculations.*—The results obtained in the determination of the field moisture equivalent of sample S 5,214 are given in Table 8. The field moisture equivalent is computed by means of the formula

$$F.\ M.\ E. = \frac{\text{weight of water}}{\text{weight of oven-dried soil}} \times 100$$

Thus in Table 8 the field moisture equivalent of sample S 5,214 equals

$$\frac{5.79}{14.12} \times 100 = 41.0 \text{ per cent.}$$

TABLE 8.—*Determination of field moisture equivalent of sample S 5,214*

Weight of glass and wet soil, grams	32.08
Weight of glass and dry soil, grams	26.29
Weight of glass, grams	12.17
Weight of water, grams	5.79
Weight of dry soil, grams	14.12
Field moisture equivalent, per cent	41.0

SHRINKAGE DETERMINATION

1. *Scope.*—This procedure furnishes the data from which the following subgrade soil constants may be computed: (*a*) Shrinkage limit, (*b*) shrinkage ratio, (*c*) volumetric change, (*d*) lineal shrinkage, and (*e*) specific gravity (approximate). Shrinkage determinations made on sample S 5,214 are recorded in Table 9.

2. *Apparatus.*—The apparatus consists of the following:

A porcelain evaporating dish about 4½ inches in diameter.
A flexible spatula having a blade about 3 inches long and about three-fourths inch wide.
A circular porcelain milk dish having a flat bottom and being about 1¾ inches in diameter by about one-half inch high.
A steel straightedge about 12 inches long.
A glass cup about 2 inches in diameter and about 1 inch high, the top rim of which is ground smooth and level.
A glass plate with three metal prongs for immersing the soil pat in mercury, as shown in Figure 10.
A glass graduate having a capacity of 25 cubic centimeters and graduated to 0.2 cubic centimeter.
An analytical balance sensitive to 0.001 gram.
Sufficient mercury to fill the glass cup to overflowing.

3. *Sample.*—A sample weighing about 30 grams is taken from the thoroughly mixed portion of the material passing the No. 40 sieve.

4. *Procedure.*—The sample is placed in the evaporating dish and thoroughly mixed with water in amount sufficient to fill the soil voids completely and to make the soil pasty enough to be readily worked into the porcelain milk dish without the inclusion of air bubbles. The amount of water required to furnish friable soils with the desired consistency is equal to or slightly greater than the liquid limit, and the amount necessary to furnish plastic soils with the desired consistency may exceed the liquid limit by as much as 10 per cent. The inside of the porcelain milk dish is coated with a thin layer of vaseline or some other heavy grease to prevent the adhesion of the soil to the dish.

An amount of the wetted soil equal to about one-third the volume of the milk dish is placed in the center of the dish, and the soil is caused to flow to the edges by tapping the dish on a firm surface cushioned by several layers of blotting paper or similar material. An amount of soil is added approximately equal to the first portion, and the dish is tapped until the soil is thoroughly compacted and all included air is brought to the surface. More soil is added and the tapping is continued until the dish is completely filled and excess soil stands out about its edge. The excess soil is then struck off with a straightedge, and all soil adhering to the outside of the dish is wiped off.

The dish when filled and struck off is weighed immediately and the weight recorded as the weight of dish and wet soil (29.34 grams, Table 9). The soil pat is allowed to dry in air until the color of the pat turns from dark to light. It is then oven-dried to constant weight at 110° C. and the weight recorded as the weight of dish and dry soil (22.61 grams, Table 9). The weight of the empty dish (11.52 grams, Table 9) is determined and recorded. The capacity of the dish in cubic centimeters, which is also the

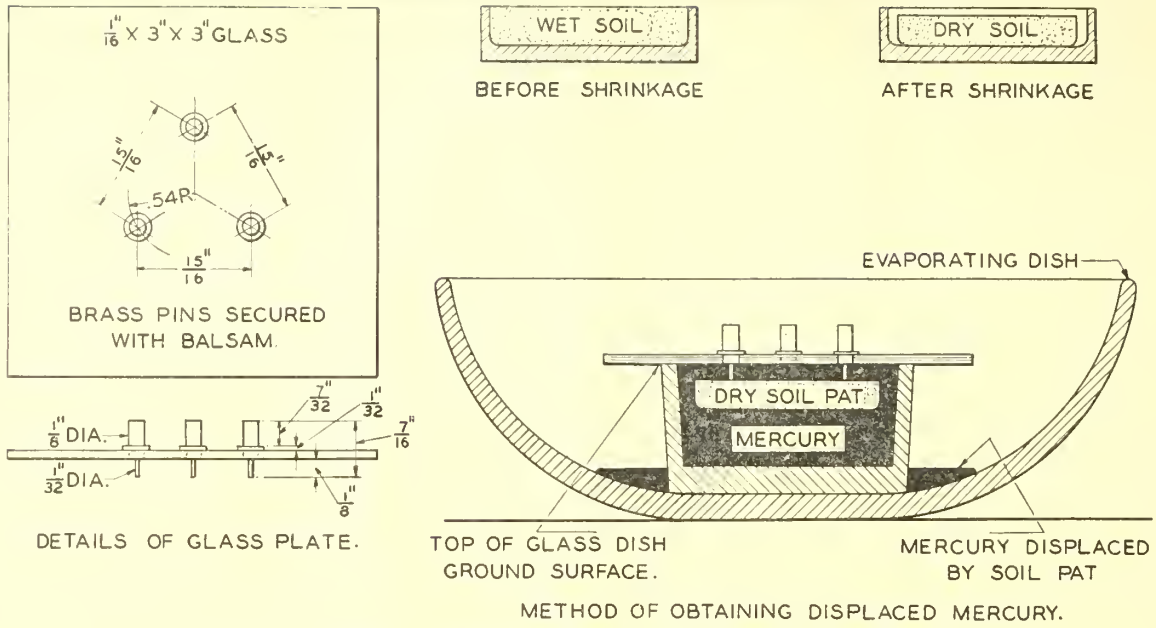

FIGURE 10.—APPARATUS FOR DETERMINING THE VOLUMETRIC CHANGE

volume of the wet soil pat, is determined by filling the dish to overflowing with mercury, removing the excess by pressing a glass plate firmly over the top of the dish, and measuring the volume of mercury held in the dish in the glass graduate. This volume is recorded as the volume of the wet soil pat, V (10.99 cubic centimeters, Table 9).

The volume of the dry soil pat is determined by removing the pat from the porcelain milk dish and immersing it in the glass cup full of mercury in the following manner: The glass cup is filled to overflowing with mercury and the excess mercury is removed by pressing the glass plate with the three prongs (fig. 10) firmly over the top of the cup. Any mercury which may be adhering to the outside of the cup is carefully wiped off. The cup, filled with mercury, is placed in the evaporating dish, and the soil pat is placed on the surface of the mercury. It is then carefully forced under the mercury by means of the glass plate with the three prongs (fig. 10) and the plate is pressed firmly over the top of the cup. It is essential that no air be trapped under the soil pat. The volume of the mercury so displaced is measured in the glass graduate and recorded as the volume of the dry soil pat, V_o (5.60 cubic centimeters, Table 9).

5. *Computations.*—The weight of the milk dish is subtracted from the weight of dish and wet pat to give the weight of the wet soil pat, W. The weight of the milk dish subtracted from the weight of dish and dry pat gives the weight of the dry soil pat, W_o. The moisture content w of the soil at the time it was put in the dish, expressed as a percentage of the dry weight of the soil, is computed from the formula

$$w = \frac{W - W_o}{W_o} \times 100$$

Thus in Table 9

$W = 29.34 - 11.52 = 17.82$ grams,

$W_o = 22.61 - 11.52 = 11.09$ grams,

and

$$w = \frac{17.82 - 11.09}{11.09} \times 100 = 60.7 \text{ per cent.}$$

CALCULATION OF SHRINKAGE LIMIT

1. *Definition.*—The shrinkage limit of a soil is that moisture content, expressed as a percentage of the weight of the oven-dried soil, at which a reduction in moisture content will not cause a decrease in the volume of the soil mass, but at which an increase in moisture content will cause an increase in the volume of the soil mass.

2. *Computations.*—The shrinkage limit, S, is calculated from the data obtained in the volumetric shrinkage determination by the following formula:[3]

$$S = w - \left(\frac{V - V_o}{W_o} \times 100 \right)$$

Thus in Table 9 the shrinkage limit of sample S 5,214 equals

$$60.7 - \left(\frac{10.99 - 5.60}{11.09} \times 100 \right) = 12.1 \text{ per cent}$$

3. *Optional method.*—When both the true specific gravity, G, and the shrinkage ratio, R, are known, the shrinkage limit may be calculated from the formula:[3]

$$S = \left(\frac{1}{R} - \frac{1}{G} \right) \times 100$$

Thus in Table 9, if G and R were known first, the shrinkage limit of sample S 5,214 could be computed as follows:

$$S = \left(\frac{1}{1.98} - \frac{1}{2.60} \right) \times 100 = 12.1 \text{ per cent}$$

CALCULATION OF SHRINKAGE RATIO

1. *Definition.*—The shrinkage ratio of a soil is the ratio between a given volume change, expressed as a percentage of the dry volume, and the corresponding change in moisture content above the shrinkage limit, expressed as a percentage of the weight of the oven-dried soil. It equals the apparent specific gravity of the dried soil pat.

[3] See PUBLIC ROADS, July, 1931, p. 125.

2. *Computations.*—The shrinkage ratio, R, is calculated from the data obtained in the volumetric shrinkage determination by the following formula:[3]

$$R = \frac{W_o}{V_o}$$

Thus in Table 9 the shrinkage ratio of sample S 5,214 equals

$$\frac{11.09}{5.60} = 1.98$$

CALCULATION OF VOLUMETRIC CHANGE

1. *Definition.*—The volumetric change of a soil for a given moisture content is the volume change, expressed as a percentage of the dry volume, suffered by the soil mass when the moisture content is reduced from the stipulated percentage to the shrinkage limit. This stipulated moisture content is usually taken as the field moisture equivalent.

2. *Computation.*—The volumetric change, *V. C.*, is calculated from the data obtained in the volumetric shrinkage determination by the following formula:

$$V.\ C. = (w_1 - S)R$$

where w_1 is the given moisture content.

If, as is customary, the volumetric change from the field moisture equivalent is desired, the formula assumes the form,

C_f = volumetric change from field moisture equivalent[4] $= (F.\ M.\ E. - S)\,R.$

Thus in Table 9 the volumetric change of sample S 5,214, when mixed with an amount of water equal to the field moisture equivalent (see Table 8) equals $(41.0 - 12.1) \times 1.98 = 57.2$ per cent.

CALCULATION OF LINEAL SHRINKAGE

1. *Definition.*—The lineal shrinkage of a soil for a given moisture content is the decrease in one dimension, expressed as a percentage of the original dimension, suffered by the soil mass when the moisture content is reduced from an amount equal to the field moisture equivalent to the shrinkage limit.

2. *Computation.*—The lineal shrinkage, *L. S.*, is obtained either by means of the formula[4]

$$L.\ S. = 100\left(1 - \sqrt[3]{\frac{100}{C_f + 100}}\right)$$

or by means of the curve shown in Figure 11, which represents this relation. Thus in Table 9 the lineal shrinkage of sample S 5,214, as obtained from the formula, is 14.0 per cent.

Table 9.—*Volumetric shrinkage determination for sample S 5,214*

Weight of dish and wet soil, grams	29.34
Weight of dish and dry soil, grams	22.61
Weight of dish, grams	11.52
W, weight of wet soil pat, grams	17.82
W_o, weight of dry soil pat, grams	11.09
w, moisture content of wet soil pat, per cent	60.7
V, volume of dish, volume of wet soil pat, cubic centimeters	10.99
V_o, volume of dry soil pat, cubic centimeters	5.60
S, shrinkage limit, per cent	12.1
R, shrinkage ratio	1.98
C_f, volumetric change from field moisture equivalent, per cent	57.2
L. S., lineal shrinkage, per cent	14.0
G, specific gravity (approximate)	2.60

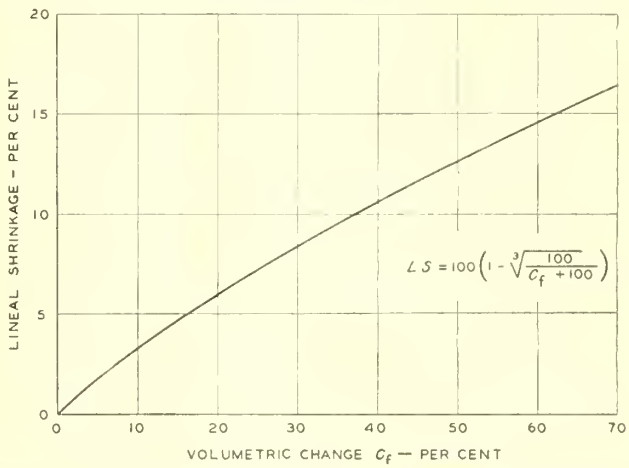

Figure 11.—Relation Between Volumetric Change and Lineal Shrinkage

APPROXIMATE CALCULATION OF SPECIFIC GRAVITY

1. *Definition.*—The specific gravity of a soil is the weight of the oven-dried soil divided by the true volume of the soil particles.

2. *Computations.*—The specific gravity, G, is calculated from the data obtained in the volumetric shrinkage test by the following formula:[4]

$$G = \frac{1}{\frac{1}{R} - \frac{S}{100}}$$

Thus in Table 9 the approximate specific gravity of sample S 5,214 is obtained by the computation

$$G = \frac{1}{\frac{1}{1.98} - \frac{12.1}{100}} = \frac{1}{0.384} = 2.60$$

[3] See Public Roads, July, 1931, p. 125.

[4] See Public Roads, July, 1931, pp. 130 and 131.

GRAPHICAL SOLUTION OF THE DATA FURNISHED BY THE HYDROMETER METHOD OF ANALYSIS

Reported by E. A. WILLIS, Assistant Highway Engineer, F. A. ROBESON, Junior Highway Engineer, and C. M. JOHNSTON, Junior Civil Engineer, United States Bureau of Public Roads

THE HYDROMETER method of mechanical analysis described previously in this issue of PUBLIC ROADS includes the following two separate operations: (*a*) The determination of the percentage of dispersed soil particles remaining in suspension at a given time, as indicated by particular hydrometer readings, and (*b*) the determination of the maximum size of soil particles in suspension corresponding to the percentages represented by particular hydrometer readings. The percentage of particles in suspension is determined by the hydrometer reading corrected for the conditions under which the test is performed. The maximum grain size at the time of any particular hydrometer reading is computed by means of Stokes's law. The method of obtaining the data is described in the preceding article (pp. 197 to 207 of this issue), Procedures for Testing Soils for the Determination of the Subgrade Soil Constants.

PERCENTAGE DETERMINATION

The Bouyoucos hydrometer, like any other, depends upon the density of the suspending medium for its buoyancy. It is calibrated in grams of soil per liter of suspension for assumed conditions of temperature of suspension and specific gravity of soil grains. Consequently, the actual number of grams per liter for any particular case is given directly by the hydrometer reading only if the conditions are identical with those for which the hydrometer was calibrated. For any other conditions suitable corrections must be made. For example, the hydrometer referred to in this discussion has been calibrated for a suspension temperature of 67° F. and a specific gravity of soil particles of 2.65. For temperatures other than 67° F. and specific gravities other than 2.65, the weight of soil remaining in suspension, in grams per liter of suspension, can be obtained by adding a temperature correction to the hydrometer reading and multiplying the sum by a specific gravity correction. The weight thus obtained is expressed as a percentage of the weight of soil originally dispersed. Both weights are given in grams of soil per liter of suspension, but, since the volume of the suspension is ordinarily one liter, the qualifying phrase, "per liter of suspension," is omitted for convenience in the discussion which follows.

When the temperature of the suspension, the specific gravity of the soil particles, and the weight of soil originally dispersed are known, the percentage of soil remaining in suspension for a given hydrometer reading may be determined graphically by the use of a chart laid out on cross-section paper, as illustrated in Figure 1. On this chart ordinates denote hydrometer readings and abscissas denote percentages of soil in suspension. Since the hydrometer is calibrated to give a correct reading in grams per liter when the temperature is 67° F. and the specific gravity is 2.65, these values are used as the base, or standard, to which the graphical corrections are referred. The value 50 grams was chosen as the basic value for the weight of soil originally dispersed. The relation between hydrometer reading and percentage of soil in suspension established by these three standard values is shown as a broken line in Figure 1.

The equation relating the hydrometer reading to the percentage of soil in suspension may be expressed as follows:

Let

R = hydrometer reading.

W = weight of soil originally dispersed per liter of suspension.

w = weight of soil in suspension per liter of suspension.

ΔR = correction to hydrometer reading for variation in temperature from 67° F.

a = correction coefficient for variation in specific gravity from 2.65.

$$= \frac{2.6500-0.9984}{2.6500} \times \frac{G}{G-0.9984},$$ where G is the specific gravity of the soil.[1]

P = percentage of originally dispersed soil remaining in suspension.

For the standard values of temperature and specific gravity (67° F. and 2.65),

$$w = R$$

For other values,

$$w = (R + \Delta R)a$$

The percentage of soil in suspension is given by the equation

$$P = \frac{(R+\Delta R)a}{W} \times 100 \qquad (1)$$

This equation may be written

$$P = (R+\Delta R) \times \frac{a}{W} \times 100 \qquad (1)$$

For the basic or standard conditions we have

$$\Delta R = 0,\ a = 1,\ W = 50,$$

so that the basic relation is given by the equation

$$P = \frac{R}{50} \times 100 = 2R$$

The manner in which the chart is constructed and the method of applying the various corrections are explained in the following paragraphs.

Temperature correction.—In Table 1 are given the values of the correction, ΔR, for variations in temperature from 67° F., obtained experimentally for the hydrometer used as an example in this discussion. If in equation (1) we put $P = 0$ we have

$$R = -\Delta R$$

It follows, therefore, that if the line represented by equation (1) should be plotted on the chart (fig. 1) its intercept on the axis of ordinates would be the point 0, $-\Delta R$. In the lower left-hand corner of Figure 1 values of ΔR taken from Table 1 are laid off as ordinates corresponding to values of the temperature from 60° to 90° F. A zero ordinate corresponds to a temperature of 67° F. and positive values of ΔR are plotted downward. By means of this scale the intercept, $-\Delta R$, for any given temperature of suspension, is readily obtained.

It will be noted in equation (1) that the slope of the line is given by the expression $\frac{a}{W} \times 100$. It is evident

[1] See preceding article, Procedures for Testing Soils for the Determination of the Subgrade Soil Constants, p. 200 of this issue.

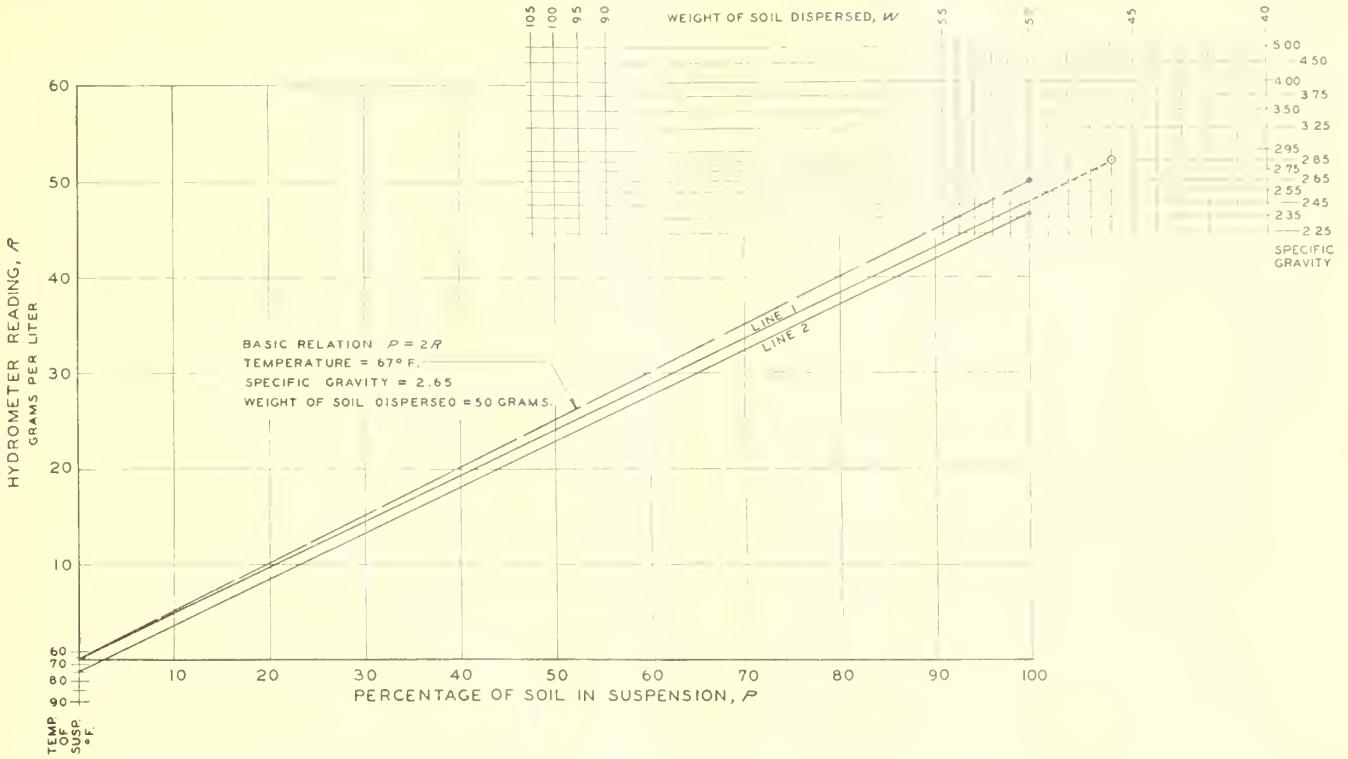

FIGURE 1.—CHART FOR CONVERTING HYDROMETER READINGS TO PERCENTAGES OF ORIGINALLY DISPERSED SOIL REMAINING IN SUSPENSION

TABLE 1.—*Temperature corrections to be added to the hydrometer readings*

Suspension temperature	Correction, ΔR
Degrees F.	*Grams per liter*
60	−0.8
65	−0.3
67	0.0
70	+0.4
75	+1.2
80	+2.2
85	+3.2
90	+4.3

that variations in the relation between P and R caused by variations in the specific gravity and the weight of soil dispersed are cared for by changes in the slope of the line.

Correction for specific gravity.—Assuming the basic values, $\Delta R = 0$, $W = 50$, we have, from equation (1)

$$P = \frac{Ra}{50} \times 100 = 2Ra$$

Setting $P = 100$, we have

$$R = \frac{50}{a} \qquad (2)$$

Equation (2) gives the value of the hydrometer reading at 100 per cent suspension for any given value of specific gravity as indicated by the value of a. In Table 2 are given values of the quantity $\frac{50}{a}$ and also of the quantity $\frac{50}{a} - 50$, the correction which must be added to or subtracted from the basic value 50 to obtain the correct reading for 100 per cent suspension.

In the upper right-hand corner of Figure 1, values of the quantity $R = \frac{50}{a}$ are laid off as ordinates corresponding to values of the specific gravity from 2.25 to 5.00. Thus, for a specific gravity of 2.65 the ordinate is 50; for a value of 5.00 the ordinate is 64.2, etc. When the weight of soil dispersed is 50 grams the slope of the line represented by equation (1) is obtained by drawing a line through the origin intersecting the abscissa $P = 100$ at the value of R corresponding to the given specific gravity.

TABLE 2.—*Values of specific gravity constant, a, $\frac{50}{a}$, and $\frac{50}{a} - 50$ for different values of the specific gravity*

Specific gravity of soil particles, G	Specific gravity constant, a	$\frac{50}{a}$	Hydrometer correction for reading of 50
		Grams per liter	*Grams per liter*
2.25	1.1201	44.6	−5.4
2.35	1.0836	46.1	−3.9
2.45	1.0519	47.5	−2.5
2.55	1.0243	48.8	−1.2
2.65	1.0000	50.0	0.0
2.75	0.9785	51.1	+1.1
2.85	0.9593	52.1	+2.1
2.95	0.9421	53.1	+3.1
3.25	0.8996	55.6	+5.6
3.50	0.8720	57.3	+7.3
3.75	0.8494	58.9	+8.9
4.00	0.8306	60.2	+10.2
4.50	0.8009	62.4	+12.4
5.00	0.7787	64.2	+14.2

Correction for weight of soil dispersed.—Assuming the basic values, temperature = 67° F. and specific gravity of soil = 2.65; i. e., $\Delta R = 0$, $a = 1$, we have, in equation (1)

$$P = \frac{R}{W} \times 100$$

Setting $R = 50$, we have

$$P = \frac{5{,}000}{W} \qquad (3)$$

Equation (3) gives the percentage of soil in suspension for a hydrometer reading of 50 and a weight of dis-

persed soil, W. Values of the quantity $\frac{5{,}000}{W}$ are given in Table 3 for values of W between 40 and 55 grams and between 90 and 105 grams. The lower interval is for use in the hydrometer analysis of silt and clay soils, the higher interval for use in the case of sandy soils. Values of the quantity $\frac{5{,}000}{W}$ are plotted as abscissas, in the upper right-hand portion of Figure 1, for indicated values of the weight of soil dispersed, W. Thus, as in Table 3, 50 grams of soil dispersed corresponds with 100 per cent of soil in suspension, 100 grams corresponds with 50 per cent, etc. When the specific gravity of the soil is 2.65, the slope of the line represented by equation (1) is obtained by drawing a line through the origin intersecting the ordinate R=50 at the value of P corresponding to the given value of W.

TABLE 3.—*Values of the quantity* $\frac{5{,}000}{W}$, *giving the percentage of particles in suspension indicated by a hydrometer reading of 50 for different weights of dry soil originally dispersed. (Specific gravity=2.65; temperature of suspension=67° F.)*

Weight of dry soil dispersed, W, per liter of suspension	Percentage in suspension, P, for hydrometer reading of 50	Weight of dry soil dispersed, W, per liter of suspension	Percentage in suspension, P, for hydrometer reading of 50
Grams		*Grams*	
40.0	125.0	90.0	55.6
41.0	122.0	91.0	54.9
42.0	119.1	92.0	54.3
43.0	116.3	93.0	53.8
44.0	113.7	94.0	53.2
45.0	111.1	95.0	52.6
46.0	108.7	96.0	52.1
47.0	106.4	97.0	51.5
48.0	104.2	98.0	51.0
49.0	102.0	99.0	50.5
50.0	100.0	100.0	50.0
51.0	98.0	101.0	49.5
52.0	96.2	102.0	49.0
53.0	94.3	103.0	48.5
54.0	92.6	104.0	48.1
55.0	90.9	105.0	47.6

For given values of temperature of suspension, specific gravity of soil, and weight of soil originally dispersed per liter of suspension, the line represented by equation (1), giving the relation between hydrometer reading and percentage of soil in suspension, is laid out on the chart of Figure 1 by the following process:

1. A straightedge is placed on the chart so as to intersect the origin and a point whose abscissa corresponds with the given weight of soil dispersed and whose ordinate corresponds with the given specific gravity, as indicated by the scales shown in the upper right-hand corner of Figure 1.

2. The straightedge is then moved parallel to its original position until it intersects the axis of ordinates at a point corresponding to the given temperature of suspension, as indicated by the temperature scale in the lower left-hand corner of Figure 1. A line is then drawn which represents the required relation between P and R.

To illustrate this operation, let us assume that 46.0 grams of soil having a specific gravity of 2.85 were dispersed and that the temperature of the suspension remained constant at 75° F. Line 1 in Figure 1, intersecting the origin and a point whose coordinates correspond with 46.0 grams of soil dispersed and a specific gravity of 2.85, represents the first position of the straightedge. Line 2, which intersects the axis of ordinates at a point corresponding to 75° F., gives the desired relation between P and R.

Values given by line 2 may be checked by computations based on equation (1). From Tables 1 and 2 we obtain the values, $\Delta R = +1.2$ and $a = 0.9593$.

Substituting in equation (1), we have

$$P = (R+1.2)\,\frac{0.9593}{46.0}\times 100$$

$$= (R+1.2)\ 2.086$$

If $P=0$, $R=-1.2$. If $R=32.0$, $P=69.3$ These values will be found to check with those given by line 2.

DETERMINATION OF GRAIN SIZE

The second distinct operation in the graphical solution of the data furnished by the hydrometer analysis consists of determining the maximum grain size in suspension at any given time from Stokes's law, which is expressed by the formula

$$d = \sqrt{\frac{30nL}{980\,(G-G_1)\,T}} \qquad (4)$$

Where

d = maximum grain diameter in millimeters.
n = coefficient of viscosity of the suspending medium, in poises.
L = distance in centimeters through which soil particles settle.
T = time in minutes, period of sedimentation.
G = specific gravity of soil particles.
G_1 = specific gravity of the suspending medium.

In order that Stokes's law may serve to disclose the diameter of the soil particles it is necessary to know the distance through which these particles fall in a given time. As has been explained previously in the report, Procedures for Testing Soils for the Determination of the Subgrade Soil Constants, the distance, L, through which the soil particles are assumed to settle, in the determination of the grain size by means of the Bouyoucos hydrometer, equals 0.42 of the total distance between the surface of the suspension and the elevation of the bottom of the hydrometer.

When the hydrometer reading, the temperature of the suspension, the period of sedimentation, and the specific gravity of the soil particles are known, the particle diameter may be determined graphically by the use of a chart constructed as shown in Figure 2. The chart is plotted on semilogarithmic cross-section paper, on which the ordinates denote hydrometer reading and the abscissas denote period of sedimentation.

In the preceding article (p. 201 of this issue) variations in L, n, and G (equation 4) were cared for by means of correction coefficients applied to values of d computed on the basis of the standard conditions, temperature = 67° F., $G=2.65$, and $L=32.5$ centimeters. The graphical method here described does not involve the use of the standard value, $L=32.5$ centimeters. Values of L as determined by actual measurements of the hydrometer in use are substituted in the equation, and recourse to a correction factor is eliminated. The manner in which variations of L, n, and G are cared for is described in the following paragraphs.

Variations in the factor L.—Solving equation (4) (Stokes's law) for T, the period of sedimentation, we have

$$T = \frac{30nL}{980\,(G-G_1)\,d^2} \qquad (5)$$

Values of T as a function of L, the distance through which the soil settles, and d, the maximum diameter of

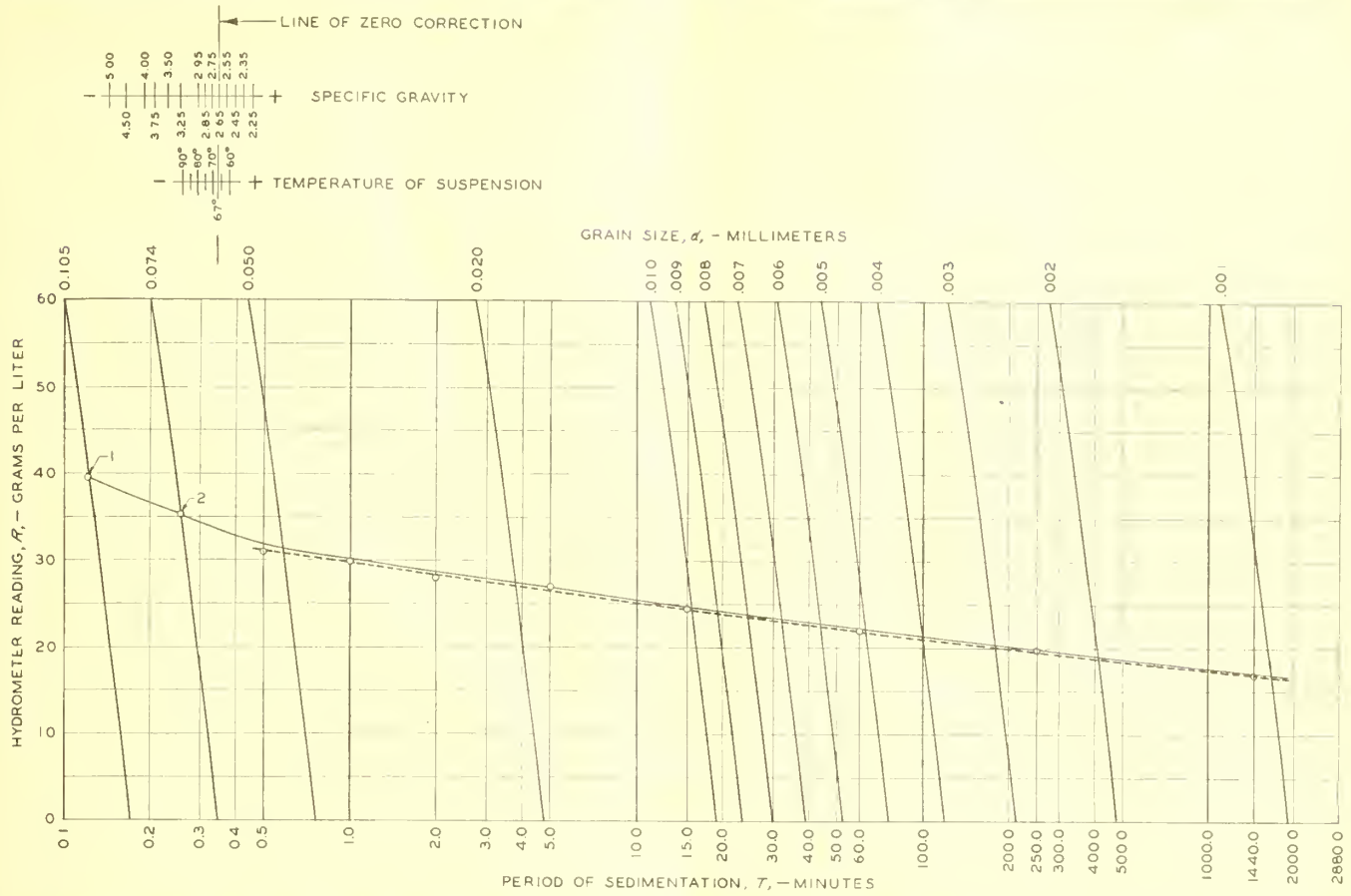
LINE OF ZERO CORRECTION
SPECIFIC GRAVITY
TEMPERATURE OF SUSPENSION
GRAIN SIZE, d, - MILLIMETERS
HYDROMETER READING, R, - GRAMS PER LITER
PERIOD OF SEDIMENTATION, T, - MINUTES

FIGURE 2.—CHART FOR OBTAINING FROM TEST DATA THE HYDROMETER READINGS CORRESPONDING TO SPECIFIC GRAIN SIZES

TABLE 4.—*Time of sedimentation as a function of grain diameter, d, and hydrometer reading, R. (Specific gravity=2.65; temperature of suspension=67° F.)*

Hydrometer reading, R	L	Time, T, in minutes for the following diameters—													
		0.105 millimeters	0.074 millimeters	0.050 millimeters	0.020 millimeters	0.010 millimeters	0.009 millimeters	0.008 millimeters	0.007 millimeters	0.006 millimeters	0.005 millimeters	0.004 millimeters	0.003 millimeters	0.002 millimeters	0.001 millimeters
Grams per liter	*Centimeters*														
0	10.00	0.171	0.345	0.756	4.73	18.9	23.3	29.5	38.6	52.5	75.6	118.0	210	473	1,891
10	9.25	.159	.319	.700	4.37	17.5	21.6	27.3	35.7	48.6	70.0	109.0	194	437	1,749
20	8.47	.145	.292	.641	4.00	16.0	19.8	25.0	32.7	44.5	64.1	100.0	178	400	1,601
30	7.78	.133	.269	.588	3.68	14.7	18.2	23.0	30.0	40.9	58.8	91.9	163	368	1,471
40	7.07	.121	.244	.535	3.34	13.4	16.5	20.9	27.3	37.1	53.5	83.5	149	334	1,337
50	6.37	.109	.220	.482	3.01	12.0	14.9	18.8	24.6	33.5	48.2	75.3	134	301	1,204
60	5.75	.099	.199	.435	2.72	10.9	13.4	17.0	22.2	30.2	43.5	67.9	121	272	1,087

soil particle remaining in suspension, are given in Table 4. The values of L to be used were determined by measurement for hydrometer readings from 0 to 60. The following quantities were assumed to be constant:

$n=0.0102$, the coefficient of viscosity of water at 67° F.
$G=2.65$.
$G_1=0.9984$, the density of water at 67° F.

The significance of Table 4 may be made clear by consideration of specific values. Thus, if the hydrometer reading, after a sedimentation period of 0.269 minutes, has the value 30, the maximum diameter of soil particles remaining in suspension is 0.074 millimeters. If, on the other hand, it takes 30 minutes for the hydrometer reading to reach the value 30, the maximum diameter of soil particles is 0.007 millimeters.

The values given in Table 4 are plotted in Figure 2 as curves showing the relation between hydrometer reading and period of sedimentation for different values of d, the maximum grain size. The relation between hydrometer reading and maximum grain size for a hydrometer analysis in which the temperature of suspension was 67° F. and the specific gravity of the soil was 2.65 may be obtained by plotting hydrometer readings against periods of sedimentation, as given by the analysis, on the chart of Figure 2. Intersections of the curve so determined with the grain-diameter curves give the hydrometer readings corresponding to specific values of maximum diameter of soil in suspension.

Variations in temperature and specific gravity.—Equation (5) may be written

$$T=\frac{30L}{980d^2}\times n\times\frac{1}{G-G_1} \quad \text{----------} \quad (5)$$

For a temperature of 67° F. and a specific gravity of 2.65, we have

$$T_o = \frac{30L}{980d^2} \times 0.0102 \times \frac{1}{2.6500 - 0.9984}$$

so that

$$\frac{T}{T_o} = \frac{n}{0.0102} \times \frac{1.6516}{G - G_1}$$

Let

$$C_n = \frac{n}{0.0102}$$

and

$$C_G = \frac{1.6516}{G - G_1}$$

$$= \frac{1.65}{G - 1} \text{ (approximately).}$$

Then

$$T = T_o \times C_n \times C_G,$$

and

$$\text{Log } T = \text{Log } T_o + \log C_n + \log C_G \text{ ..... (6)}$$

The grain-diameter curves plotted in Figure 2 give correct values of T, the period of sedimentation, for the standard conditions, temperature = 67° F. and $G = 2.65$. To obtain correct values for other temperatures and specific gravities, it is necessary to multiply all values of T by the factor $C_n \times C_G$. Since the scale of T is logarithmic, it is evident that the correction can be applied by displacing the entire system of curves horizontally a distance equal to the algebraic sum of the corrections, $\log C_n + \log C_G$. If this sum is positive, the curves should be shifted to the right; if negative, to the left.

Values of the viscosity coefficient of water, n, the temperature correction $C_n = \frac{n}{0.0102}$, and $\log_{10} C_n$ are given in Table 5 for temperatures varying from 60° to 90° F. Values of C_n are plotted against temperature of suspension in the upper left-hand corner of Figure 2 to the same logarithmic scale as that to which the period of sedimentation is plotted on the main chart. The reference line (temperature = 67° F., $C_n = 1.00$, $\log C_n = 0$) may be taken at any convenient point. Distances along this correction scale may be laid off directly from the logarithmic scale by the use of dividers. For construction purposes the actual lengths may be computed accurately by multiplying each value of $\log C_n$ by the length of one cycle on the chosen logarithmic scale.

TABLE 5.—*Values of the viscosity coefficient,[1] n, of water at various temperatures, of the coefficient* $C_n = \frac{n}{0.0102}$, *and of log* $_{10}C_n$.

Temperature	n	Coefficient C_n	$\text{Log}_{10} C_n$
° F.	*Poises*		
60	0.0112	1.10	0.04139
65	.0105	1.03	0.01284
67	.0102	1.00	0.00000
70	.00978	0.959	−0.01818
75	.00917	0.899	−0.04624
80	.00861	0.844	−0.07366
85	.00810	0.794	−0.10018
90	.00764	0.749	−0.12552

[1] Smithsonian Physical Tables, seventh revised edition, 1921.

Similarly, values of C_G and $\log_{10} C_G$ are given in Table 6 for specific gravities varying from 2.25 to 5.00, and a logarithmic scale of these corrections is shown above the temperature correction scale in Figure 2.

As stated above, the grain-diameter curves may be corrected for given values of temperature and specific gravity by horizontal displacement in the direction indicated by the sign of the correction. To plot a point representing a given hydrometer reading and a given period of sedimentation, it is necessary to apply the correction in the opposite direction, i. e., positive to the left. The relative position of the point with respect to the curves is then the same as if the curves had been shifted to the right (or to the left, in the case of a negative correction).

TABLE 6.—*Values of the specific gravity correction,* $C_G = \frac{1.65}{G-1}$, *and of log* $_{10}C_G$, *for values of the specific gravity, G, from 2.25 to 5.00*

Specific gravity, G	Coefficient, C_G	$\text{Log}_{10} C_G$
2.25	1.320	0.1206
2.35	1.222	0.0872
2.45	1.138	0.0561
2.55	1.064	0.0272
2.65	1.000	0.0000
2.75	0.943	−0.0256
2.85	0.892	−0.0497
2.95	0.846	−0.0726
3.25	0.733	−0.1347
3.50	0.660	−0.1805
3.75	0.600	−0.2218
4.00	0.550	−0.2596
4.50	0.471	−0.3266
5.00	0.412	−0.3846

METHOD OF USING CHARTS DESCRIBED

In Table 7 are given the data obtained in the hydrometer analysis of a soil sample. The following paragraphs describe the steps involved in using the charts of Figures 1 and 2 to develop from this material the data on which the grain-size accumulation curve is based. The results thus obtained are given in Table 8.

TABLE 7.—*Sieve and hydrometer analysis of sample 5,394X*

A.—GENERAL DATA

Weight of air-dried sample, grams	49.0
Weight of dry soil dispersed, grams	46.0
Specific gravity	2.85
Plasticity index	18.0

B.—HYDROMETER TEST DATA[1]

Date tested	Time observed	Temperature	Hydrometer reading	Period of sedimentation
		° F.	*Grams per liter*	*Minutes*
June 25, 1930	9.30 a. m.			0
Do	9.30.5 a. m.	76	31.0	0.5
Do	9.31 a. m.	75	30.0	1
Do	9.32 a. m.	75	28.0	2
Do	9.35 a. m.	75	27.0	5
Do	9.45 a. m.	75	24.5	15
Do	10.00 a. m.	74	24.0	30
Do	10.30 a. m.	74	22.0	60
Do	1.40 p. m.	74	20.0	250
June 26, 1930	9.30 a. m.	73	17.0	1,440

[1] Volume of suspension, 1 liter.

C.—SIEVE ANALYSIS

Fraction	Weight	Percentage of dispersed sample
	Grams	
Retained on No. 10		0
Passing No. 10, retained on No. 20	0.98	2.13
Passing No. 20, retained on No. 40	0.80	1.74
Passing No. 40, retained on No. 60	0.90	1.96
Passing No. 60, retained on No. 140	4.32	9.39
Passing No. 140, retained on No. 200	3.80	8.26

1. On tracing paper placed over Figure 1, line 2 is constructed by the method previously described, for a specific gravity, G, of 2.85, a weight of dry soil dispersed, W, of 46.0 grams, and a temperature of suspension of 75° F., as indicated in Tables 7, A and 7, B. While some of the temperatures listed in Table 7, B vary slightly from 75° F., the temperature throughout the test may be assumed as being 75° F.

2. The percentages of material retained on each of the sieves in the sieve analysis are computed and listed in Table 7, C. From these data the percentage of material smaller than each of the sieve sizes is computed and recorded in the last column of Table 8.

3. On a piece of tracing paper placed over Figure 2 a curve of time against hydrometer reading is plotted, as shown by the broken line in Figure 2, from the observed data as recorded in Table 7, B. The intersections of this curve with the grain-diameter curves give the hydrometer readings corresponding to the times of sedimentation indicative of the grain sizes represented by the grain-diameter curves, uncorrected for variation of the specific gravity of the soil particles in suspension and the temperature of the suspension from the standard values, 2.65 and 67° F., respectively.

4. The tracing paper is shifted horizontally through a distance equal to the algebraic sum of the temperature and specific gravity corrections, which in this case, for a temperature of 75° F. and a specific gravity of 2.85, is to the right. It should be noted that moving the tracing paper upon which the curve of time against hydrometer reading has been plotted produces the same effect as moving the grain-diameter curves in the opposite direction. It is for this reason that a point representing the algebraic sum of the specific gravity and temperature corrections, easily obtained by means of a pair of dividers, is laid off from the line of zero correction in the direction corresponding to the sign of that sum and the tracing paper moved until this point coincides with the line of zero correction.

This curve in its shifted position, shown as a full line in Figure 2, gives correctly the hydrometer readings corresponding to the grain sizes indicated by the grain-diameter curves.

5. The hydrometer readings corresponding to the percentages of material passing the No. 140 and the No. 200 sieves as recorded in Table 8 are obtained from line 2, Figure 1. In this case the values are 84.78 per cent and 76.52 per cent, respectively, and their computed hydrometer readings are 39.5 and 35.3. These hydrometer readings are plotted on the grain-diameter curves (fig. 2) corresponding to their respective sieve sizes with the tracing paper in its shifted position. These points are shown as points 1 and 2, Figure 2.

6. The two points are then connected with the curve previously drawn (full line, fig. 2) in order to tie in the results given by the sieve analysis with those given by the hydrometer analysis.

7. The hydrometer readings corresponding to the grain sizes tabulated in Table 8 are obtained from the curve (full line, fig. 2). Thus the hydrometer reading corresponding to a grain diameter of 0.05 millimeter is 32.0 and the hydrometer reading corresponding to a grain diameter of 0.002 millimeter is 19.3.

8. For these hydrometer readings the corresponding percentages of soil in suspension are obtained from line 2 (fig. 1). Thus a hydrometer reading of 32.0 indicates 69.3 per cent of soil in suspension and a hydrometer reading of 19.3 indicates 42.8 per cent of soil in suspension. These values are recorded in Table 8 and used in plotting the soil accumulation curve, which is shown in Figure 3. From this curve the percentage of soil smaller than any specific diameter may be obtained.

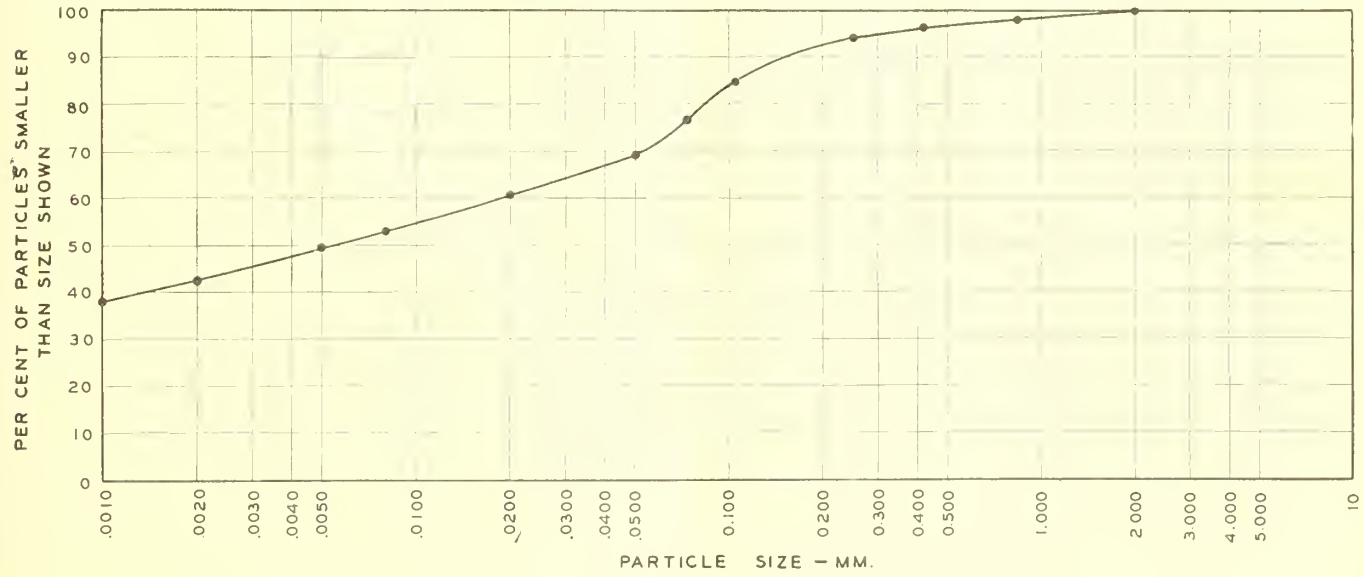

FIGURE 3.—GRAIN-SIZE ACCUMULATION CURVE FOR SOIL SAMPLE 5,394X

TABLE 8.—*Grain-size accumulation data for soil sample 5,394X*

Sieve No.	Maximum grain size of fraction	Hydrometer reading	Percentage of total sample
	Millimeters	*Grams per liter*	
10	2.0	----------	100.00
20	0.84	----------	97.87
40	0.42	----------	96.13
60	0.25	----------	94.17
140	0.105	[1] 39.5	84.78
200	0.074	[1] 35.3	76.52
	0.05	32.0	69.3
	0.02	28.0	61.0
	0.008	24.4	53.4
	0.005	22.5	49.4
	0.002	19.3	42.8
	0.001	17.2	38.4

[1] Computed.

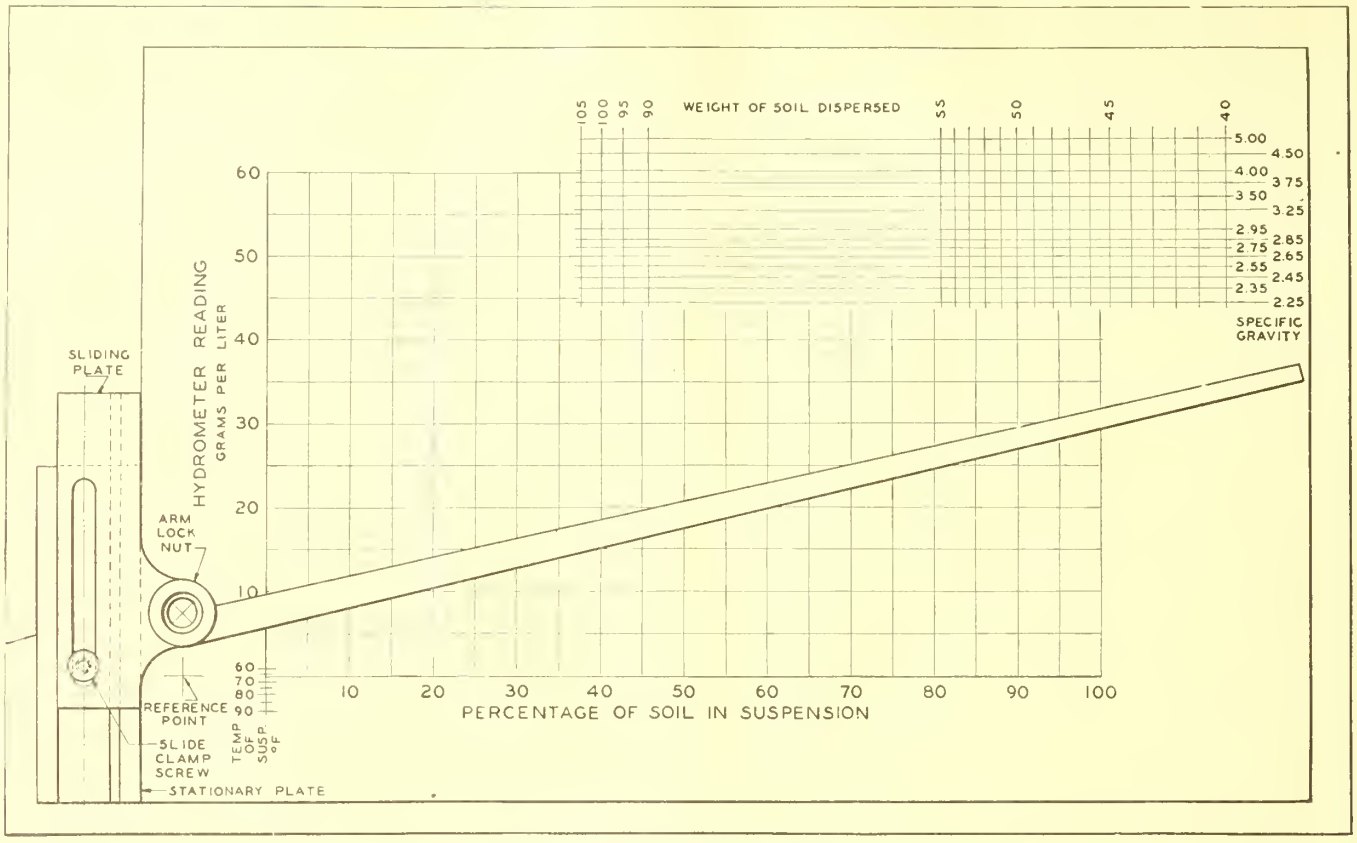

FIGURE 4.—CHART AND PROTRACTOR FOR CONVERTING HYDROMETER READINGS TO PERCENTAGES OF ORIGINALLY DISPERSED SOIL REMAINING IN SUSPENSION

SPECIAL APPARATUS ASSISTS IN PERFORMING THE GRAPHICAL SOLUTION

The foregoing procedure was described in detail in order to explain clearly the basis of the various operations. The procedure used in the laboratory of the Bureau of Public Roads is the same as that described above in essentials, but is somwhat simplified by the use of special apparatus.

The apparatus which assists in determining the relation between the hydrometer reading and the percentage of soil in suspension is shown in Figure 4. It consists of a chart containing scales similar to those shown in Figure 1 and an adjustable protractor which slides on a fixed base. Both the protractor and the chart are mounted on a drawing board 12 inches wide and 16 inches long.

The scales for hydrometer reading, percentage of soil in suspension, temperature, and specific gravity are constructed exactly like and in the same position as the same scales in Figure 1. A reference point for the fulcrum of the protractor arm is located 1 inch to the left of the origin. To compensate for this displacement, the scale for weight of soil dispersed is shifted 1 inch to the left of the position shown in Figure 1. This displacement was necessitated by the fact that if the reference point were at the origin the fulcrum of the protractor would interfere with reading the scales in the lower left-hand corner of the chart.

The protractor is operated in the following manner: The slide clamp screw is loosened and the protractor is moved vertically until the intersection of the cross hairs in the transparent center of the fulcrum of the protractor is directly over the reference point. The slide clamp screw is then tightened and the arm lock nut is loosened. The protractor arm is rotated until its reading edge passes through a point whose abscissa corresponds to the given weight of soil displaced and whose ordinate corresponds to the given specific gravity. The arm lock nut is then tightened, the slide clamp screw is loosened, and the protractor is moved vertically until the reading edge of the arm intersects the temperature scale at the desired point. The slide clamp screw is then tightened. In this position the reading edge of the protractor arm, corresponding in position to line 2 (fig. 1), will give correctly the percentage of soil particles in suspension indicated by the hydrometer readings.

The apparatus which assists in determining the grain size is illustrated in Figure 5. It consists of a chart similar to that shown in Figure 2, but without the temperature and specific gravity scales, and a sliding carriage which serves to shift the tracing paper, all mounted on a drawing board 16 by 21 inches in size. The tracing paper is held in the sliding carriage by two clamps. The sliding of the carriage is controlled by means of a thumbscrew and thrust yoke located at the upper right-hand corner of the board.

The specific gravity scale is scribed on the carriage exactly as it was plotted in Figure 2, and the temperature scale is scribed on a fixed metal guide adjacent to the specific gravity scale. The scale values are the same as were used in the construction of the temperature scale (fig. 2), but positive values are plotted to the left and negative values to the right of the zero correction line. This is necessary because the specific gravity scale moves with the tracing paper but the temperature scale remains fixed. The position of these scales on the apparatus is shown in Figure 5. The chart containing the grain-size curves is held in place on the drawing board by means of two clamps, as shown.

This apparatus is operated in the following manner: The tracing paper is inserted in the slot in the movable carriage provided for it and the clamps are tightened. By means of the adjusting screw the position of the carriage is adjusted so that a specific gravity of 2.65 on the specific gravity scale coincides with a temperature of 67° F. on the temperature scale. A curve of time of sedimentation against hydrometer reading is plotted on the tracing paper in the manner previously described. The tracing paper is then displaced horizontally by moving the slide carriage until the two correction scales are in such a position that the given specific gravity coincides with the given temperature. The curve of time of sedimentation against hydrometer reading will then be in the correct position to give the hydrometer readings corresponding to the grain diameters for which the information is desired.

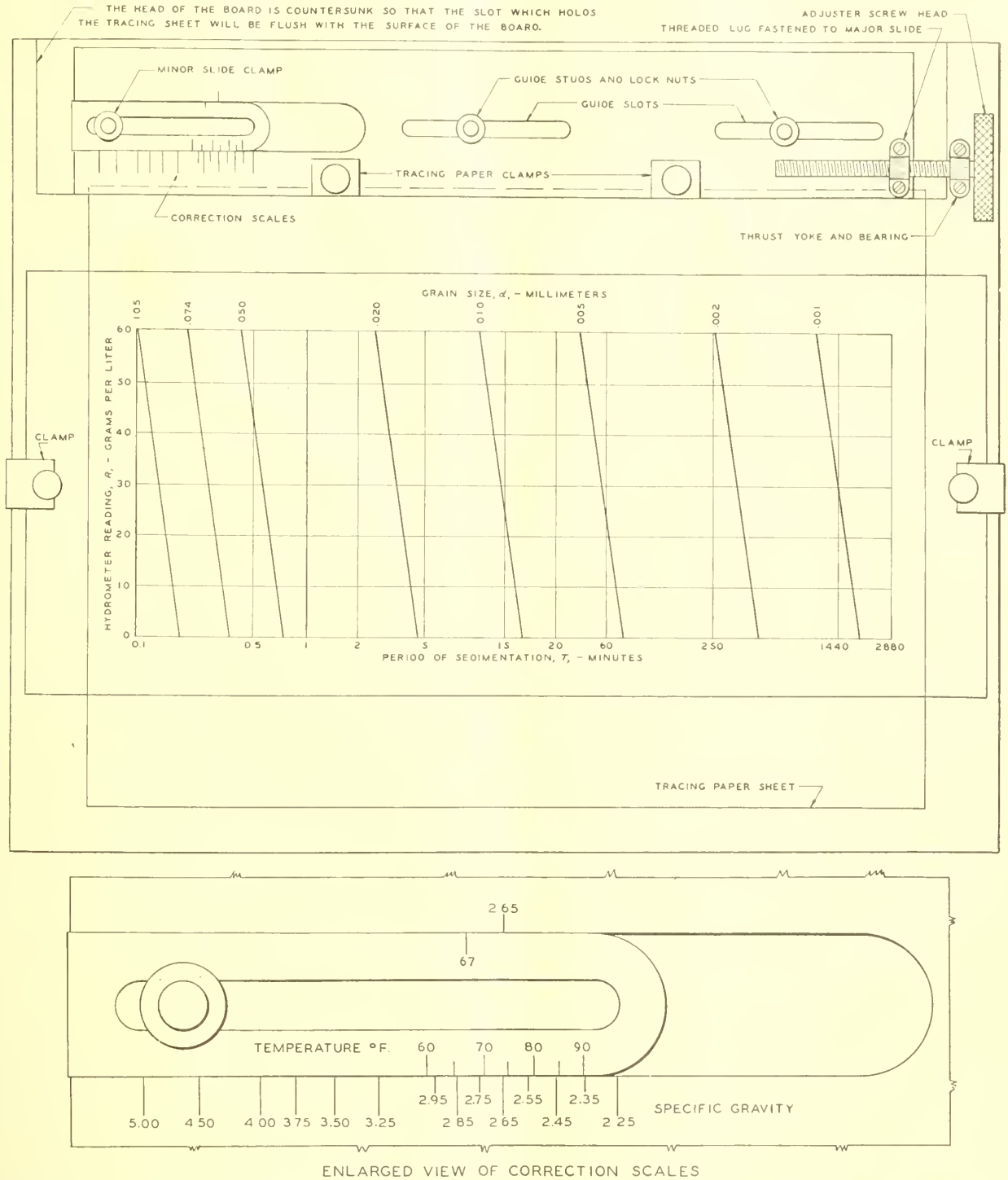

FIGURE 5.—CHART AND MECHANICAL DEVICE FOR OBTAINING FROM TEST DATA THE HYDROMETER READINGS CORRESPONDING TO SPECIFIC GRAIN SIZES

INVESTIGATION OF TAR ROADS IN PROGRESS

In cooperation with representatives of the tar industry, the Bureau of Public Roads is conducting an investigation of low cost and secondary roads on which tar was used in construction and maintenance. A committee of three was appointed to represent the producers of tar, and an engineer was assigned to work directly with members of the bureau staff in making the study. Work on the project was begun in the early part of this year.

Although tar has been used as a road material in different parts of the country for many years, it is of special interest at the present time to study its performance when used in low-cost road treatments. It is believed that the investigation which is now being carried on will produce valuable information regarding methods of construction and maintenance costs, and the behavior of tar-bound surfaces under light and medium traffic.

The procedure adopted involves (*a*) a preliminary survey of the type of road selected for study, and (*b*) a detailed study of a limited number of road projects. The preliminary survey serves to bring out the factors which should be given attention in the special study. Roads are selected which afford an opportunity to investigate all such factors.

The detailed study of each project involves a complete investigation of the methods employed in construction, the materials and proportions used, maintenance of the road, and its surface condition. Samples are taken to determine the thickness and condition of the mat. In addition to the field work, exhaustive laboratory tests are made on the materials composing the mat, as well as on the base materials and the subgrade. The study also includes an analysis of the costs of construction and maintenance. Information regarding the volume and character of the traffic, if not available from other sources, is obtained in the course of the field work.

Since the beginning of this investigation early in 1931, a survey has been made in North Carolina of sand-clay, topsoil, traffic-bound gravel, and stone roads, surface-treated with tar. Over 1,000 miles of such surfaces have been built in the past few years and an extensive program of surface treatment is now in progress. The survey has been completed and a report is now being prepared.

A second survey, covering tar-bound roads in Pennsylvania, is now under way. The preliminary survey has been completed and the detailed study begun. In addition to a study of low-cost surface treatments, the Pennsylvania work includes an investigation of tar-bound macadam construction. Surfaces of this type consist generally of a layer of aggregate 1¼ inches to 5/8 inch in size spread to a loose depth of about 3 inches, penetrated with a cold application of tar, and compacted by rolling or by traffic.

In the Pennsylvania study it will be possible to investigate old as well as new work, thereby obtaining information on the effect of changes in methods and materials over a period of years. Following the completion of the work in Pennsylvania it is planned to extend the investigation to different parts of the country, and to study other types of construction which have proved their worth and might with advantage come into wider use. Information of value derived from these studies will be published in the form of progress reports as rapidly as possible.

TRAFFIC SURVEY OF WASHINGTON, D. C., AREA BEGUN

On September 10, 1931, the Bureau of Public Roads, in cooperation with other agencies, inaugurated a survey of traffic on the principal highways in the area immediately adjacent to Washington, D. C. The cooperating agencies are the National Capital Park and Planning Commission, the District of Columbia, the Maryland State Roads Commission, the Virginia Department of Highways, the Maryland National Capital Park and Planning Commission, the City of Alexandria, Arlington and Fairfax Counties, Virginia; and Prince Georges and Montgomery Counties, Maryland.

This is the first comprehensive survey of the area surrounding the city of Washington. It is expected to yield information essential to the solution of traffic problems in this region. The general purpose of the survey, according to the agreement signed by the cooperating agencies, is "to secure facts and obtain information which will enable the agencies concerned to develop a comprehensive, systematic, and connected system of main highways within and between the areas of Fairfax and Arlington Counties, Va.; the city of Alexandria, Va.; Montgomery and Prince Georges Counties, Md.; and the District of Columbia, including a study of proper connections with the Federal-aid highway systems of Maryland and Virginia within these areas, and such other adjacent areas as may be affected by a general highway plan for the environment of the District of Columbia. The surveys will be directed toward the development of a priority program of highway construction and betterment in the entire area and toward fixing the location and connections of suitable belt lines, relief roads, or necessary relocations and extensions of existing highways. They will include a highway traffic census to determine the origin and destination of various kinds of vehicles, the preparation of a comprehensive report, and an analysis of the information obtained, for the guidance of the officials concerned in the various jurisdictions."

INDEX TO VOLUME 10 OF PUBLIC ROADS AVAILABLE

An index to volume 10 of Public Roads, which includes the issues from March, 1929, to February, 1930, is now available for distribution, and copies may be obtained without charge from the Bureau of Public Roads, United States Department of Agriculture, Washington, D. C. Indexes to volumes 6, 7, 8, and 9 have previously been published, and a supply of these indexes is still on hand. The index to volume 11 is now being prepared.

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ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.
Report of the Chief of the Bureau of Public Roads, 1929.
Report of the Chief of the Bureau of Public Roads, 1930.

DEPARTMENT BULLETINS

No. *136D Highway Bonds. 20c.

*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.

*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.

*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.

*660D. Highway Cost Keeping. 10c.

1279D. Rural Highway Mileage, Income, and Expenditures 1921 and 1922.

1486D. Highway Bridge Location.

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No. 331C. Standard Specifications for Corrugated Metal Pipe Culverts.

TECHNICAL BULLETIN

No. 55T. Highway Bridge Surveys.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y. Road Work on Farm Outlets Needs Skill and Right Equipment.

MISCELLANEOUS CIRCULARS

No. 62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-Aid Highway Projects.

*93M. Direct Production Costs of Broken Stone. 25c.

109M. Federal Legislation and Regulations Relating to the Improvement of Federal-Aid Roads and National-Forest Roads and Trails, Flood Relief, and Miscellaneous Matters.

MISCELLANEOUS PUBLICATION

No. 76MP. The Results of Physical Tests of Road-Building Rock.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio. (1927)

Report of a Survey of Transportation on the State Highways of Vermont. (1927)

Report of a Survey of Transportation on the State Highways of New Hampshire. (1927)

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio. (1928)

Report of a Survey of Transportation on the State Highways of Pennsylvania. (1928)

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D– 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.

Vol. 5, No. 19, D– 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.

Vol. 5, No. 24, D– 6. A New Penetration Needle for Use in Testing Bituminous Materials.

Vol. 11, No. 10, D–15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

SEPTEMBER 30, 1931

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage	MILEAGE Total	Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage	MILEAGE Total		
Alabama	2,274.1	3,977,720.41	1,975,776.79	158.6	36.4	195.0						3,328,206.06	Alabama
Arizona	913.9	5,308,952.41	3,652,791.85	230.0	189.2	419.2						311,214.62	Arizona
Arkansas	1,859.1	5,341,014.54	2,793,272.77	121.7	96.9	218.6	506,930.89	253,465.43	19.6	4.9	24.5	109,241.63	Arkansas
California	1,996.3	12,179,922.30	5,267,802.87	316.9	101.3	418.2	1,227,415.38	219,892.50	41.3		41.3	299,698.19	California
Colorado	1,361.1	7,370,115.99	3,918,400.78	312.7	92.1	404.8	849,448.61	384,260.11	31.8	9.4	41.2	493,223.26	Colorado
Connecticut	268.3	4,691,129.05	1,693,759.63	43.0		43.0							Connecticut
Delaware	323.9	928,942.00	463,716.60	42.1		42.1						39,994.21	Delaware
Florida	554.4	6,181,508.65	2,872,456.15	174.4		174.4						1,609,098.91	Florida
Georgia	2,868.2	8,859,090.98	4,172,237.29	255.1	151.9	407.0	1,666,425.08	613,501.89	29.3	35.5	64.8	93,679.39	Georgia
Idaho	1,344.6	3,581,620.31	2,046,594.97	232.2	74.7	306.9	408,363.79	202,206.15	13.0	12.8	25.8	198,110.41	Idaho
Illinois	2,397.8	25,858,345.54	12,008,686.52	782.5	31.9	814.4	4,675,237.47	1,978,513.89	152.5	4.6	157.1		Illinois
Indiana	1,597.4	11,681,398.77	5,766,524.14	371.7		371.7	698,282.70	349,140.34	38.3		38.3	416,716.87	Indiana
Iowa	3,229.6	5,107,576.34	2,007,337.70	93.5	69.5	163.0						15,206.91	Iowa
Kansas	3,432.9	6,096,166.17	2,908,115.71	356.6	36.0	392.6	40,717.03	18,414.02	3.4		3.4	161,096.64	Kansas
Kentucky	1,646.8	5,261,246.41	2,421,400.07	293.1	69.9	363.0	185,744.32	94,502.33	3.9	5.8	9.7	38,946.75	Kentucky
Louisiana	1,455.0	7,899,135.56	3,675,142.63	188.6	20.3	208.9	2,567,840.55	135,581.56	1.0		1.0	57,860.55	Louisiana
Maine	651.4	3,716,399.99	1,535,875.48	82.2		82.2	474,329.91	213,859.88	3.9		3.9	325,509.16	Maine
Maryland	740.1	1,198,056.70	512,236.52	40.7		40.7	49,918.00	24,959.00	2.7		2.7	30,887.35	Maryland
Massachusetts	736.4	11,230,074.71	3,224,991.50	97.4		97.4	1,449,468.21	561,989.51	13.3		13.3	430,185.38	Massachusetts
Michigan	1,826.6	11,924,482.85	5,183,141.46	395.4	50.3	445.7	248,466.96	123,275.00	13.1		13.1	1,368,527.77	Michigan
Minnesota	3,956.1	8,465,031.03	3,701,797.71	79.0	317.7	396.7	154,382.24	33,028.32	.9	1.2	2.1	14,252.31	Minnesota
Mississippi	1,764.8	4,349,611.86	2,134,938.39	184.3	88.0	272.3	242,383.87	119,760.39		12.6	12.6	3,693,928.90	Mississippi
Missouri	2,754.9	7,488,442.66	3,121,381.06	234.7	34.6	269.3						162,198.26	Missouri
Montana	2,190.8	10,003,739.65	5,616,401.77	644.0	115.6	759.5	609,999.76	341,636.02	59.5	33.4	92.9	1,047,978.39	Montana
Nebraska	3,913.1	9,913,671.70	4,617,451.98	299.1	203.0	502.1	255,703.05	127,851.50	27.0	28.4	55.4	410,995.70	Nebraska
Nevada	1,094.6	3,262,391.58	2,138,387.70	108.6	263.9	372.5	4,951.87	4,399.73		.5	.5	197,354.49	Nevada
New Hampshire	393.9	1,584,270.81	568,311.17	32.2	7.2	39.4							New Hampshire
New Jersey	574.2	5,379,968.28	1,896,751.43	62.0	.3	62.3	1,398,428.28	538,623.71	5.6	.5	6.1	194,826.21	New Jersey
New Mexico	2,158.9	2,986,906.05	1,918,542.91	90.9	75.3	166.2	107,277.63	67,700.35	.3		.3	125,621.09	New Mexico
New York	2,764.6	37,196,891.40	13,938,669.98	725.3	7.0	732.3	1,595,300.00	560,450.00	26.2		26.2	279,475.24	New York
North Carolina	2,148.6	3,114,668.75	1,511,179.36	89.0	17.4	106.4	334,529.01	167,214.48	16.5	15.4	31.9	1,469,519.39	North Carolina
North Dakota	4,836.9	2,719,314.00	1,357,433.85	317.2	254.4	571.6	629,477.13	334,060.22	31.0	184.8	215.8	720,443.91	North Dakota
Ohio	2,613.0	16,092,046.81	5,666,137.42	278.0	39.2	317.2	1,645,404.97	605,101.03	33.6	5.7	39.3	412,305.61	Ohio
Oklahoma	2,036.0	7,103,270.25	3,631,930.70	246.6	127.6	374.2	601,125.31	157,257.22	25.5	4.0	29.5	16,649.85	Oklahoma
Oregon	1,379.3	6,053,783.66	3,386,428.56	223.1	50.1	273.2	147,885.65	88,871.50	13.3		13.3	361,504.59	Oregon
Pennsylvania	2,817.7	10,900,013.33	4,681,763.22	240.4		240.4	15,163.78	7,040.14	.1		.1	457,715.24	Pennsylvania
Rhode Island	249.4	755,371.45	293,691.92	8.2		8.2	37,587.29	18,793.64	.1		.1	160,101.56	Rhode Island
South Carolina	1,883.9	4,850,633.05	2,157,898.52	78.2	122.3	200.5	587,639.45	150,042.23	19.7	9.0	28.7	100.50	South Carolina
South Dakota	3,913.1	4,254,340.36	2,240,765.21	226.0	243.0	469.0	65,501.32	35,061.21	.6	10.0	10.6	22,796.13	South Dakota
Tennessee	1,665.3	610,739.89	301,317.46	8.5		8.5						2,026,820.81	Tennessee
Texas	7,156.4	18,151,754.17	8,916,457.32	762.8	287.5	1,060.3	3,354,659.10	1,421,422.33	181.5	44.3	225.8	1,737,037.59	Texas
Utah	1,174.7	1,129,554.05	629,522.38	54.6	16.4	71.0	417,076.93	292,465.40	26.6	13.7	40.2	265,181.73	Utah
Vermont	303.8	1,335,410.39	536,957.67	34.9	4.7	39.6						8,133.69	Vermont
Virginia	1,717.7	4,376,715.08	2,030,600.67	198.3	25.8	224.1	1,062,329.16	527,151.20	50.1		50.1	181,577.01	Virginia
Washington	1,020.2	5,634,594.75	2,476,358.47	164.4	39.7	204.1	743,149.65	265,600.00	24.6		24.6	547,590.20	Washington
West Virginia	777.7	5,189,650.64	2,203,247.03	134.5	22.0	156.6	225,073.78	104,726.24	5.5		5.5	94,288.56	West Virginia
Wisconsin	2,460.4	8,378,010.98	3,348,706.43	266.0	84.4	350.4	285,044.61	90,909.97	10.8	.6	11.4	12,692.50	Wisconsin
Wyoming	1,817.3	3,460,525.90	1,922,106.77	247.1	187.8	434.9	39,799.12	26,128.12	8.7		8.7	72,545.77	Wyoming
Hawaii	57.8	1,058,807.25	463,088.90	33.0		33.0	160,999.13	62,206.07	1.7		1.7	1,506,890.40	Hawaii
TOTALS	93,072.0	344,072,068.46	155,608,487.28	10,659.3	3,655.2	14,314.5	29,779,361.01	11,321,052.63	936.4	437.1	1,373.5	25,517,828.57	TOTALS

[1]The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 12, NO. 9 NOVEMBER, 1931

APPLYING PRIME COAT TO SAND-CLAY BASE

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

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BITUMINOUS TREATMENTS ON SAND-CLAY AND MARL BASES IN SOUTH CAROLINA

Reported by H. C. JONES, Division Engineer, South Carolina Highway Department, and E. L. TARWATER, Assistant Highway Engineer, United States Bureau of Public Roads

A COOPERATIVE experimental project was constructed during the summer and fall of 1929 in Berkeley County, S. C., by the State highway department and the Bureau of Public Roads. This project, 4.5 miles in length, was built on State route No. 46 from its junction with United States Highway No. 17 at Moncks Corner to the village of Pinopolis.

The purpose of the experiments was to obtain information on the following subjects:

(*a*) The comparative value of sand-clay and marl as bases for bituminous surface treatment.

(*b*) The comparative value of various types of bituminous material for cold application.

(*c*) The effect of variously graded aggregates.

(*d*) Comparison of the wearing surfaces produced by the mixed-in-place method and by ordinary surface treatment.

The project embraced the construction of bituminous wearing surfaces by the mixed-in-place and surface-treatment methods, using crushed stone of different gradings and various bituminous materials. These surfaces were constructed upon sand-clay and marl bases which had been previously built.

The bituminous materials used were 8–13 viscosity tar, 60–70 and 85–100 penetration asphalts cut back with heavy naphtha, 25–35 viscosity tar, and an asphalt emulsion. The aggregate used was crushed granite graded as follows: 1¼ to ¼ inch, ¾ to ¼ inch, ⅝ to ¼ inch, and ¼ inch to dust. Table 1 gives the types and qualities of materials used in the construction of the sections. The analyses of the bituminous materials, aggregates, and mixes are given in Tables 2 to 7, inclusive.

The road upon which the experiments were built carries passenger cars and light trucks mainly but also some heavier trucks with trailers carrying relatively heavy loads of logs or merchandise. A traffic count at stations 13 and 92 taken shortly after construction showed a count of 587 and 320 vehicles daily, respectively.

The area through which the road passes is relatively low, and for this reason the water table is rather close to the surface of the ground. In spite of this condition, fair drainage of the roadway proper is provided. In several small areas where the grade is exceptionally low French drains were later constructed, extending from the edges of the treatments through the shoulders to the drainage ditches.

The marl and sand-clay bases were constructed by contract in the summer and fall of 1928. The bituminous wearing surfaces were built by State forces during the summer and fall of 1929.

CONSTRUCTION OF THE MARL BASE

Two miles of this project were constructed upon a marl base, the material for which was obtained from a local deposit about 1½ miles south of the project. Laboratory tests show it to have the following characteristics:

	Per cent
Calcium carbonate	86. 85 to 83. 85
Silica, alumina, and iron oxides	10. 40 to 14. 55
Magnesium carbonate	1. 33 to 1. 65

Cementing value, 133 to 500 plus.

As prepared for the above test it did not slake in 60 minutes.

When taken from the pit the marl is grayish white in color, contains considerable moisture, and in this condition can be easily broken down with picks or sledges and readily worked with a drag, disk, or blade machine. As it is homogeneous it compacts uniformly when moist and does not laminate but may develop small shrinkage cracks while drying or setting up. When dry it is white and in sunlight causes an objectionable glare. After it is compacted and set up it is very hard and can not be machined without wetting. It does not ravel or pothole under traffic, but does wear away, as some of

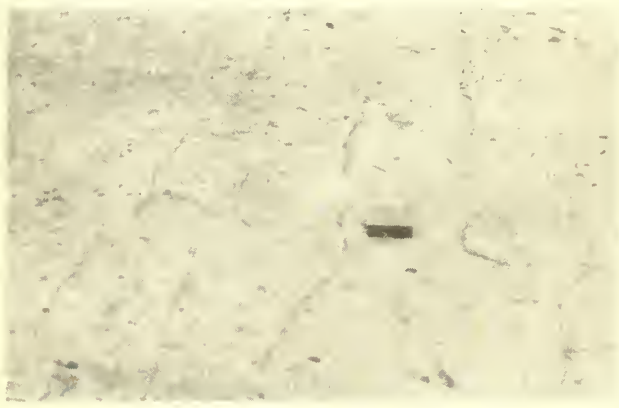

FIGURE 1.—CHARACTERISTIC SHRINKAGE CRACKS DEVELOPING IN THE MARL BASE AS IT DRIES OUT AND HARDENS

the surface particles are whipped off as dust. Ordinary rains do not affect it, but continued rains or trapped water held on the surface soften it, allowing it to rut. Where the grade is low and where the subsoil is of a type permitting capillary moisture, softening of this material occurs and results in weakening and failure of the surfacing.

Surface irregularities can not be repaired by the ordinary patching method when the marl is dry and has set up. The new material will harden but will not bond to the old surface and will eventually scale off. Scarifying and reworking seems to be the most satisfactory method of surface repair.

In constructing the marl base side forms were used and were spaced 20 to 21 feet apart except for a distance of about 500 feet through the business section of Moncks Corner where the width was made about 30 feet. The forms were set to rough grade, to give a base thickness of approximately 8 inches compacted. The marl was dumped between the forms, broken into small pieces with picks and sledges, and then worked with the disk harrow and blade machine to develop a uniform mass. The material was rolled for compaction, and during this operation blading was continued to bring the roadway to the proper grade and cross section.

The marl base was constructed in the fall of 1928, which was an unusually wet season, and considerable rutting occurred under traffic, which was compelled to use the road during construction. The surface was machined, but some areas became so badly rutted that scarifying and reworking was necessary.

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TABLE 1.—*Types and quantities of materials used in the bituminous construction*

Experiment No.	Section No.	Stations: Beginning	Stations: End	Base: Type	Base: Average depth	Primer 8–13 viscosity tar	Bituminous mat: Stone: Coarse	Bituminous mat: Stone: Fine [1]	Bituminous mat: Bituminous material: Kind	Bituminous mat: Bituminous material: Rate of application	Bituminous mat: Average thickness compacted	Seal: Bituminous material: Description	Seal: Bituminous material: Rate of application	Seal: 5/8–¼ inch stone
					Inches	*Gals. per sq. yd.*	*Lbs. per sq. yd.*	*Lbs. per sq. yd.*		*Gals. per sq. yd.*	*Inches*		*Gals. per sq. yd.*	*Lbs. per sq. yd.*
1	A	0+00	13+28	Marl	9	0.27	[2] 154	38	60–70 penetration asphalt cut-back.	0.91	1.8	60–70 penetration asphalt cut-back.	0.42	30
	B	13+28	24+50	do	10	.27	[2] 156	36.5	85–100 penetration asphalt cut-back.	1.03	1.8	85–100 penetration asphalt cut-back.	.29	15
2	A	[3] 24+50	39+60	do	8.5	.31	[2] 180		do	.69	2.0	do	.30	18
	B	39+60	52+00	do	8	.29	[2] 175		25–35 viscosity tar	.89	2.0	25–35 viscosity tar	.23	15
	C	52+00	66+00	do	9	.28	[2] 165	37	Asphalt emulsion	1.27	2.3	Asphalt emulsion	.23	15
3		79+20	92+40	do	9	.30	[4] 162	38	85–100 penetration asphalt cut-back.	1.19	1.8			
4		92+40	105+84	do	9	.26	[4] 157		do	.72	1.8	85–100 penetration asphalt cut-back.	.22	15
5		66+00	79+20	do		.33	[2] 50		do	[5] .64				
6	A	105+84	119+99	Sand-clay	7	.33	[2] 171	40	60–70 penetration asphalt cut-back.	1.00	2.0	do	.34	15
	B	119+99	132+00	do	7	.39	[2] 170	42	85–100 penetration asphalt cut-back.	1.09	2.0	do	.23	15
7	A	132+00	144+20	do	6	.29	[2] 178		do	.77	2.0	do	.27	18
	B	199+70	212+90	do		.25	[2] 169		25–35 viscosity tar	.77	2.0	25–35 viscosity tar	.32	15
	C	144+20	158+40	do		.34	[2] 165	30	Asphalt emulsion	1.11	2.3	Asphalt emulsion	.25	15
8		212+90	226+10	do		.26	[4] 154	38	85–100 penetration asphalt cut-back.	1.25	1.8			
9		226+10	238+80	do		.27	[4] 152		do	.77	1.8	85–100 penetration asphalt cut-back.	.29	14
10		[6] 158+40	199+70	do	7	.29	[2] 52 and [4] 13		do	[7] .77				

[1] Stone screenings, ¼ inch to dust.
[2] 1¼-inch–¼-inch crushed stone.
[3] Deduct 36 linear feet for railroad crossing.
[4] ¾-inch–¼-inch crushed stone.
[5] Includes retreating stations 66+00 to 73+40.
[6] Deduct 170 linear feet for correction equation.
[7] Includes original treatment and retreatment.

TABLE 2.—*Analysis of asphalt cut-backs used in the construction*

Type material	85–100 penetration	85–100 penetration	85–100 penetration	85–100 penetration	60–70 penetration	60–70 penetration	60–70 penetration
Laboratory Number	31933	32529	32530	32531	31938	3253?	32533
Sections on which used	1–B, 5, 10	1–B, 2–A, 3, 8, 10	2–A, 4, 6–A, 9, 10	3, 1, 6–B, 7–A, 10	6–A	1–A	6–A
Specific gravity, 25°/25° C.	0.942	0.947	0.943	0.945	0.947	0.950	0.949
Flash point, ° C.	29.8	28	29	25	31.4	29.60	28.4
Viscosity, Engler at 40° C.	61.58	54.49	40.32	47.71	58.52	70.58	65.61
Viscosity Engler at 50° C.	34.21	29.98	23.09	28.11	30.30	37.30	35.01
Percentage loss at 163° C., 5 hours, 50 grams	25.31	24.50	24.74	28.15	28.28	25.38	27.69
Penetration of residue, 25° C.	104	114	119	117	84	101	85
Percentage loss at 163° C., 5 hours, 20 grams	25.89	24.92	25.59	25.80	29.84	26.30	28.58
Penetration of residue, 25° C.	56	65	63	64	44	50	47
Bitumen soluble in CS_2, per cent	99.81	99.79	99.79	99.88	99.91	99.81	99.79
Organic insoluble, per cent	0.07	0.18	0.17	0.10	0.08	0.17	0.18
Inorganic insoluble, per cent	0.02	0.03	0.04	0.02	0.01	0.02	0.03
Bitumen insoluble in 86° Baumé naphtha, per cent	15.82	17.05	15.98	14.89	19.15	18.84	19.35
Residue of 100 penetration, per cent	73.9	74.7	74.9	74.4	72.9	72.1	72.4
Penetration of residue at 25° C.	101	98	104	90	94	87	89
Penetration of residue at 0° C.	12	22	22	22	14	22	23
Softening point of residue, ° C.	48.4	45.4	45	46.8	52.2	46.8	46.4
Ductility of residue at 25° C.	110+	110+	110+	110+	110+	110+	110+
Ductility of residue at 1.5° C.	6.4	5.8	4.9	6.2	5.2	4.5	4.0
Distillation test, A. S. T. M. modified D 20[1] fractions, per cent by weight:							
0–170° C.	4.24	1.08	0.25	1.27	0.42	1.06	0.05
170–225° C.	12.32	14.02	15.83	15.66	18.80	15.67	17.81
225–315° C.	6.57	5.32	8.05	6.26	8.24	7.62	8.53
315–360° C.	2.05	3.27	1.54	1.48	1.67	1.87	1.91
Residue over 360° C.	74.60	75.90	74.20	75.10	70.70	73.70	71.60
Penetration of residue at 25° C.	133	151	128	137	108	1.30	117
Softening point of residue, ° C.	40.8	43.8	42.2	42.4	44	43.8	46.6
A. S. T. M. modified D 20[2] fractions:							
0–190° C.	[3] 7.93 / [4] 10.0	[3] 0.49 / [4] 0.8	[3] 5.55 / [4] 8.2	[3] 8.04 / [4] 11.5	[3] 7.75 / [4] 9.8	[3] 4.79 / 6.02	[3] 5.52 / [4] 7.08
190–225° C.	[3] 7.31 / [4] 10.4	[3] 15.11 / [4] 19.2	[3] 10.48 / [4] 13.4	[3] 8.57 / [4] 11.0	[3] 11.44 / [4] 15.2	[3] 10.38 / [4] 12.97	[3] 11.73 / [4] 14.58
225–360° C.	[3] 8.80 / [4] 12.2	[3] 9.17 / [4] 9.7	[3] 9.81 / [4] 12.0	[3] 8.23 / [4] 9.9	[3] 9.99 / [4] 12.4	[3] 9.61 / [4] 11.60	[3] 9.95 / [4] 12.06
Penetration of residue at 25° C.	98	96	78	86	70	75	67
Softening point of residue, ° C.	44.6	47.2	46.2	47.8	49.8	48.4	50

[1] Test specimen 100 grams; thermometer ½ inch from bottom of flask; flask cooled below 125° C. before pouring residue.
[2] Test specimen 100 cubic centimeters; thermometer ¼ inch from bottom of flask; residue poured immediately after reaching 360° C.
[3] Per cent by weight.
[4] Per cent by volume.

At the time the bituminous wearing surfaces were constructed the base was, in general, in very good condition. Small shrinkage cracks had developed to a limited extent on nearly all the sections. The surface had worn unevenly in a number of areas which, after wetting, were lightly machined to restore the smooth cross section. The thickness of the marl base on the several sections was determined prior to surfacing and ranged from 7 to 10 inches, averaging about 9 inches.

CONSTRUCTION OF THE SAND-CLAY BASE

The sand-clay base upon which 2½ miles of the project were constructed was built on the old road except for a short relocated section. The sand-clay material was

obtained from local pits and, although of only fair quality for this type of construction, largely because of lack of uniformity, it was considered the best available.

TABLE 3.—*Analysis of materials used in the cut-back asphalts*

BASE MATERIAL

Material	60–70 penetration	85–100 penetration
Laboratory number	32700	32701
Specific gravity 25°/25° C	1.021	1.020
Flash point, °C	310	305
Penetration at 25° C	67	100
Softening point, °C	51.5	47.4
Ductility at 25° C., centimeters	110+	110+
Percentage loss at 163° C., 5 hours, 50 grams	.01	.02
Penetration of residue at 25° C	58	82
Bitumen soluble in CS_2, per cent	99.88	99.88
Organic matter insoluble, per cent	.09	.09
Inorganic matter insoluble, per cent	.03	.03

DISTILLATE

Laboratory number	32702
Specific gravity, 25°/25° C	0.786
Flash point, °C	0.6

Distillation:	°C.
Initial boiling point	78
First 10 cubic centimeters over at	134
Second 10 cubic centimeters over at	154
Third 10 cubic centimeters over at	167
Fourth 10 cubic centimeters over at	174
Fifth 10 cubic centimeters over at	179
Sixth 10 cubic centimeters over at	183
Seventh 10 cubic centimeters over at	187
Eighth 10 cubic centimeters over at	192
Ninth 10 cubic centimeters over at	201
End point	272

	Cubic centimeters
Distillate	99.4
Residue	.1
Loss	.5

TABLE 4.—*Analysis of tar used in the construction*

Grade of material	8–13 viscosity		8–13 viscosity	
Laboratory number	31939		32534	
Sections on which used	1–A, 1–B, 5, 6–A, 6–B, 7–A, and 10		2–A, 2–B, 2–C, 3, 4, 7–B, 7–C, 8, and 9	
Material used as	Primer.		Primer.	
Specific gravity, 25°/25° C	1.142		1.148	
Viscosity, Engler at 40° C	11.74		11.05	
Bitumen soluble in CS_2, per cent	96.93		96.15	
Organic matter insoluble, per cent	2.65		3.81	
Inorganic matter insoluble, per cent	.02		.04	
Water, per cent	.40			
Distillation, water-free basis				
Fractions:	Character	Per cent by weight	Character	Per cent by weight
Up to 170° C	Fluid	1.25	Fluid	1.05
170°–235° C	do	6.93	Trace of solids.	6.67
235°–270° C	do	11.89	½ solid	11.57
270°–300° C	¼ solid	8.40	Fluid	8.45
Residue		71.49		72.10
Softening point of residue, °C		43.2		42.8

Grade of material	25–35 viscosity		25–35 viscosity	
Laboratory number	32504		32528	
Sections on which used	2–B and 7–B		2–B and 7–B	
Material used as	Mix.		Mix.	
Specific gravity, 25°/25° C	1.165		1.165	
Viscosity, Engler at 40° C	28.52		27.49	
Bitumen soluble in CS_2, per cent	96.98		95.92	
Organic matter insoluble, per cent	2.04		3.61	
Inorganic matter insoluble, per cent	.02		.07	
Water, per cent	.96		.40	
Distillation, water-free basis				
Fractions:	Character	Per cent by weight	Character	Per cent by weight
Up to 170° C	Fluid	0.53	Fluid	1.07
170°–235° C	¼ solid	5.29	do	4.71
235°–270° C	⅔ solid	8.89	½ solid	10.26
270°–300° C	½ solid	8.03	Trace of solids.	7.89
Residue		77.26		76.07
Softening point of residue, °C		41.8		41.4

Test results on samples taken from the finished sand-clay base are given in Table 8. In the construction the sand-clay from the pit was spread on the roadway, mixed with the surface material by means of disks and plows, and consolidated by traffic. In general the sand-clay failed to bond readily and necessitated considerable machining after rains. In a few areas where this material continually failed to bond it was necessary to plow and remix and, in some cases, to add additional material.

The sand-clay base prior to the construction of the bituminous surfacing was variable in appearance and composition. Some portions were well bonded and apparently in a suitable condition for surface treatment, while other portions showed a decided tendency to dust and ravel. On some areas where patching had been done, the patches peeled off for lack of bond. Attempts were made to better the surface condition by dragging but, on account of lack of uniformity in the underlying material, this effort did not entirely correct the defects. In general, the sand-clay base was in only fair condition when the experiments were built.

SIDE FORMS USED FOR BOTH TYPES OF CONSTRUCTION

In the construction of the mixed-in-place sections on both the marl and sand-clay bases side forms were used except where otherwise stated. Bridge plank, 3 inches by 8 inches by 16 feet long, laid flatwise, were used. They were laid loosely in place but were held to line by stakes. Before the stone was spread for mixing they were moved in about a foot on either side. The

TABLE 5.—*Analysis of asphalt emulsion*

Laboratory number	32505
Section on which used	2–C and 7–C
Specific gravity 25°/25° C	1.009
Viscosity, Engler, at 50° C	1.98
Distillation to 260° C.:	
Water, per cent by weight	47.3
Oil	Trace
Residue, per cent by weight	52.6
Tests on residue from distillation:	
Specific gravity, 25°/25° C	1.015
Penetration at 25° C	113
Softening point, °C	47.8
Ductility at 25° C. (centimeters)	96.5
Bitumen soluble in CS_2, per cent	98.94
Organic matter insoluble, per cent	.40
Ash (by ignition), per cent	.66

TABLE 6.—*Grading of the crushed stone used in the construction*

Laboratory number	31906	32520	31946	31947
Sections on which used	1–A, 1–B, 2A, 2–B, 2–C, 5, 6, 7–A, 7–B, 7–C, 10	1–A, 1–B, 3, 6–A, 6–B, 8	3, 4, 8, and 9	2–A, 2–B, 2–C, 4, 7–A, 7–B, 7–C
Size, inches	1¼ to ¼	¼ to dust	¾ to ¼	⅝ to ¼

SIEVE ANALYSIS

Passing—	Retained on—			
		Per cent	Per cent	Per cent	Per cent
1¼-inch	1-inch	16			
1 inch	¾-inch	23			
¾-inch	½-inch	24		6	1
½-inch	¼-inch	25	0.1	53	70
¼-inch	10-mesh	12	15.3	29	25
10-mesh	20-mesh		32.5	7	3
20-mesh	30-mesh		11.2	5	1
30-mesh	40-mesh		11.1		
40-mesh	50-mesh		8.5		
50-mesh	80-mesh		10.4		
80-mesh	100-mesh		3.1		
100-mesh	200-mesh		4.9		
200-mesh			2.9		

TABLE 7.—*Analysis of bituminous aggregates*

Laboratory number	Experiment and section No.	Sampled: Date (1929)	Sampled: Time	Date tested (1929)	Sieve analysis of mix: Bitumen, per cent	1¼-inch to 1-inch	1-inch to ¾-inch	¾-inch to ½-inch	½-inch to ¼-inch	¼-inch to 10-mesh	10-mesh to 40-mesh	40-mesh to 80-mesh	80-mesh to 200-mesh	Passing 200-mesh	Percentage lost on heating at 60° C. (based on percentage of bitumen): 1 day	5 days	10 days	Remarks
31934	1-A	Aug. 17	Upon completion of mixing.	Nov. 4	2.2	13.0	25.2	28.7	16.8	2.8	5.5	4.3	0.2	1.3	4.76	6.09	7.41	
32515	1-A	Sept. 19	After opening to traffic.	Nov. 11	3.0	.....	6.4	34.6	26.5	9.0	10.8	4.1	2.9	2.7	.....	.....	.....	Sampled before sealing.
31905	1-B	Aug. 5	Upon completion of mixing.	Nov. 4	2.5	8.6	31.1	26.0	13.3	4.3	7.3	5.1	.2	1.6	2.21	3.41	4.81	
32516	1-B	Sept. 19	After opening to traffic.	Nov. 11	3.8	.....	10.3	44.8	16.4	7.1	8.9	3.6	2.5	2.6	.....	.....	.....	Sampled after sealing.
32524	2-A	Aug. 27	Upon completion of mixing.	Nov. 4	2.6	.....	32.3	28.0	28.4	4.5	1.6	1.3	.2	1.1	17.15	20.79	22.42	
32506	2-A	Sept. 19	After opening to traffic.	Nov. 11	2.3	16.9	25.2	19.2	21.0	7.7	3.1	1.5	1.2	1.9	.....	.....	.....	Sampled before sealing.
32523	2-B	Aug. 27	Upon completion of mixing.	Nov. 5	2.2	[1] 11.3	24.5	21.4	22.6	7.6	1.9	1.4	.2	1.1	20.54	21.31	22.59	
32507	2-B	Sept. 19	After opening to traffic.	Nov. 12	3.4	13.5	17.5	19.0	23.6	9.6	5.2	2.3	2.7	3.2	.....	.....	.....	Do.
32508	2-C	...do	...do	...do	4.8	2.7	6.2	15.2	26.8	22.7	12.2	3.6	3.3	2.5	.....	.....	.....	Sampled after sealing.
32525	3	Aug. 24	Upon completion of mixing.	Nov. 5	3.8	.....	.....	8.1	49.8	20.1	10.3	5.9	.2	1.8	5.67	7.01	7.68	
32511	3	Sept. 19	After opening to traffic.	Nov. 12	3.7	.....	.....	6.0	43.8	23.9	13.2	4.0	2.7	2.7	.....	.....	.....	No seal used.
32522	4	Aug. 23	Upon completion of mixing.	Nov. 5	1.6	.....	.....	13.7	55.7	22.2	3.4	2.0	.2	1.2	8.41	9.46	12.16	
31936	6-A	Aug. 15	...do	Nov. 6	1.6	25.6	34.4	13.4	10.0	3.2	5.6	4.5	.2	1.2	7.57	8.23	10.45	
32513	6-A	Sept. 19	After opening to traffic.	Nov. 13	4.2	.....	12.4	31.8	17.2	9.5	12.4	5.2	3.9	3.4	.....	.....	.....	Sampled after sealing.
31935	6-B	Aug. 14	Upon completion of mixing.	Nov. 6	2.8	20.8	24.4	20.2	13.3	5.0	7.1	5.1	.2	1.1	2.54	3.41	4.92	
32514	6-B	Sept. 19	After opening to traffic.	Nov. 13	2.6	3.3	10.7	41.6	15.9	9.7	7.3	4.2	2.6	2.1	.....	.....	.....	Sampled before sealing.
31937	7-A	Aug. 14	Upon completion of mixing.	Nov. 6	1.9	6.6	47.9	27.7	9.6	1.2	1.5	2.5	.2	.9	2.74	4.42	5.47	
32512	7-A	Sept. 19	After opening to traffic.	Nov. 13	2.9	.....	17.6	43.6	16.6	5.7	3.8	4.1	2.9	2.8	.....	.....	.....	Sampled after sealing.
32526	7-B	Sept. 7	Upon completion of mixing.	Nov. 7	1.6	8.2	26.8	30.8	18.8	5.3	2.4	4.4	.2	1.5	7.57	11.49	15.16	
32527	8	Aug. 31	...do	...do	3.5	.....	.....	8.4	49.8	19.5	10.0	6.7	.2	1.9	6.95	9.37	10.25	
32509	8	Sept. 19	After opening to traffic.	Nov. 14	4.1	.....	.....	6.9	34.0	23.1	14.7	7.9	6.0	3.3	.....	.....	.....	No seal used.
32521	9	...do	Upon completion of mixing.	Nov. 7	2.4	.....	.....	12.8	56.3	21.4	3.8	2.2	.2	.9	6.92	11.49	12.77	
32510	10	...do	After opening to traffic.	Nov. 14	5.8	2.0	5.6	17.4	17.2	11.3	8.2	15.9	11.2	5.4	.....	.....	.....	Sampled before sealing.

[1] Sample contained 5.8 per cent passing the 1½-inch sieve and retained on the 1¼-inch.

TABLE 8.—*Analysis of sand-clay base*

Laboratory number	Location: Experiment	Location: Station	Mechanical analysis [1]: Percentage of particles having diameters smaller than— 2 millimeters	0.5 millimeter	0.25 millimeter	0.05 millimeter	0.005 millimeter	0.001 millimeter	Liquid limit	Plasticity index	Shrinkage: Limit	Shrinkage: Ratio	Moisture equivalent: Centrifuge	Moisture equivalent: Field	Subgrade group [2]
31915	6-A	113+43	99	93	82	30	19	15	26	12	17	1.8	16	22	A-2
31911	6-A	117+60	99	92	75	29	19	14	25	9	17	1.8	14	19	A-2
31909	6-B	107+54	99	89	70	27	16	11	21	8	17	1.8	19	17	A-2
31913	6-B	120+29	99	92	75	30	21	15	28	12	18	1.8	20	23	A-2
31908	6-B	124+06	100	90	70	24	16	12	22	8	.....	.....	14	19	A-2
31910	6-B	128+64	99	89	71	29	19	15	24	10	18	1.8	19	20	A-2
31916	7-A	135+15	100	92	78	18	11	7	17	0	.....	.....	9	16	A-3
31912	7-A	139+49	99	89	69	24	17	14	24	10	17	1.8	14	20	A-2
31914	7-A	139+96	100	93	79	20	12	10	19	5	.....	.....	13	17	A-2
32517	7-B	201+00	99	95	80	22	14	11	20	7	18	1.8	13	16	A-1
32519	7-B	205+00	99	95	80	22	14	11	21	7	17	1.8	16	17	A-1
32518	7-B	209+00	100	95	73	18	10	7	18	0	.....	.....	11	16	A-3
31901	10	166+50	100	95	80	30	21	16	30	16	18	1.8	18	19	A-2
31902	10	187+00	100	95	83	21	11	8	17	0	.....	.....	9	17	A-3
31903	10	193+00	100	95	85	28	17	14	23	9	17	1.8	15	19	A-2
31904	10	198+00	100	96	88	23	15	12	22	8	.....	.....	13	18	A-2

[1] Particles above 0.10 millimeter in diameter by sieve method, Particles below 0.10 millimeter in diameter by hydrometer method.

[2] The classification of subgrade soils in groups is discussed in the article, Subgrade Soil Constants, Their Significance, and Their Application in Practice, Public Roads, vol. 12, No. 4, June, 1931, pp. 105–108. The characteristics of the 3 groups represented here are described as follows:

"*Group A-1.*—Well graded material, coarse and fine, excellent binder. Highly stable under wheel loads, irrespective of moisture conditions. Functions satisfactorily when surface treated or when used as a base for relatively thin wearing courses.

"*Group A-2.*—Coarse and fine materials, improper grading or inferior binder. Highly stable when fairly dry. Likely to soften at high water content caused either by rains or by capillary rise from saturated lower strata when an impervious cover prevents evaporation from the top layer, or to become loose and dusty in long-continued dry weather.

"*Group A-3.*—Coarse material only, no binder. Lacks stability under wheel loads but is unaffected by moisture conditions. Furnishes excellent support for flexible pavements of moderate thickness and for relatively thin rigid pavements."

stone was then roughly leveled to the forms, the bituminous material applied, and the mixing operation carried out. Prior to spreading the mix the forms were set back in place and left there during the spreading, shaping, and rolling, after which they were removed. Immediately after their removal the shoulders were built up with earth to protect the edges of the treatments.

Five experiments, in which various bituminous materials and grades of aggregate were used, were constructed upon the marl base and duplicate experiments were built upon the sand-clay base. Some of the experiments were subdivided so that the project embraces a total of 16 sections, each of which is described in detail in the following pages.

LEFT: APPEARANCE OF SAND-CLAY SECTION DURING MACHINING AND COMPACTION UNDER TRAFFIC PRIOR TO CONSTRUCTING THE BITUMINOUS SURFACE

BELOW: SAND-CLAY BASE PRIMED AND READY FOR THE STONE TO BE SPREAD

ABOVE: SAND-CLAY SURFACE PRIOR TO TREATMENT. SIDE TRENCHES CUT TO OBTAIN A THICKENED EDGE

RIGHT: TO PREVENT SEGREGATION, WHERE BOTH COARSE AND FINE STONE WERE USED IN A MIX, THE COARSE AGGREGATE WAS SPREAD FIRST AND GIVEN AN APPLICATION OF BITUMINOUS MATERIAL, AFTER WHICH THE FINE STONE WAS SPREAD AS SHOWN.

FIGURE 2.—PREPARATION FOR MIXED-IN-PLACE CONSTRUCTION ON SAND-CLAY BASE

EXPERIMENTAL SECTIONS CONSTRUCTED ON MARL BASE

Prior to the construction of the wearing surface, the marl base was thoroughly swept and given a prime coat of 8–13 viscosity tar applied at a temperature of 125° to 155° F. The amount used on each section is given in the text and in Table 1 and the analysis in Table 4. The tar was not absorbed by the marl as quickly as by the sand-clay, but a penetration of about ¼ inch was obtained. The primed surface presented a uniform, dull black appearance except for a few spots where clay balls appeared. These remained bright and glossy as the tar did not penetrate the dense clay. It did, however, penetrate around the clay so that upon its removal a completely primed surface was exposed. On unshaded areas traffic seemed not to affect the primed surface nor did the machine mixing appear to damage it to any great extent. On some areas which were partly shaded the prime coat penetrated more slowly and did not form such a firm crust, possibly because of the higher percentage of moisture present in the marl in these shaded areas. Traffic over these areas picked up the primed surface, necessitating considerable repriming by hand. Hauling over these portions of the primed surfaces was discontinued and during construction the stone was distributed by backing the trucks in over that previously spread. The wearing surfaces were not constructed until the primer had penetrated and thoroughly dried.

MIXED-IN-PLACE SECTIONS

Experiment 1, section B, stations 13+28 to 24+50.—The method of construction and the amounts of material per square yard used on this section were as follows:

Mix: 1.8 inches, compacted; 156 pounds of 1¼ to ¼ inch crushed stone, 36.5 pounds of ¼ inch to dust, and 1.03 gallons of 85–100 penetration asphalt cut back with naphtha.

Seal: 0.29 gallon of the same bituminous material was applied and covered with 15 pounds of ⅝ to ¼ inch stone chips.

Section 1–B was the first of the mixed sections constructed and was necessarily more experimental in character than the others. Considerable latitude was permitted by the outline, which did not specify the exact details of construction. As might be expected, some methods tried out were unsatisfactory and were modified or discarded.

The tar prime coat was applied at the rate of 0.27 gallon per square yard.

One hundred and fifty-six pounds of the 1¼ to ¼ inch stone per square yard were cast between the forms from stock piles at the sides and then leveled off with a drag, after which a single application of 0.46 gallon per square yard of cut-back asphalt was spread. Thirty-six and one-half pounds of the fine aggregate per square yard were then spread as uniformly as possible with shovels and followed by an application of 0.57 gallon of the same bituminous material.

Mixing was begun immediately with a heavy 4-way road drag pulled by a 10-ton tractor but the results obtained were not satisfactory. The amount of aggregate was more than the drag could handle, and when the blades were raised to lighten the load considerable material was necessarily dropped, so that after a few trips with the drag it was distributed very nonuniformly throughout the section. A 12-foot road machine was then substituted for the drag and was entirely satisfactory for the mixing operation. Because of the unevenness of the surface caused by the drag, it was impossible to redistribute the mix to a uniform thickness with the blade grader, and for this reason considerable work was done by hand and with trucks during the spreading operation. The rate of mixing on this section was somewhat slow due to the inexperience of the operators, with the result that mixing was not completed during the first day and the partly mixed material was left windrowed in the center over Sunday. By Monday morning a hard crust had formed on the outside of the windrow but during the mixing it broke up readily. Final shaping and smoothing was done very successfully with the drag. However, some handwork was necessary, particularly along the edges. This was required on practically all of the mixed sections.

During the mixing operation there was very littl tendency for the aggregates to segregate or for the fine aggregate to ball up. Because of the stiffness of the mix, some segregation did occur during the final shaping of the section, with the result that the center 7 or 8 feet presented a more porous and open appearance than the remainder of the surface.

Rolling with a 10-ton 3-wheel roller was begun immediately after final shaping was completed. As the operators were inexperienced and the roller was not in good mechanical condition, the rolling was not prosecuted as speedily or as efficiently as desirable but was carried on intermittently for several days. A reasonable degree of surface smoothness was obtained, but it was evident that better compaction would have resulted had the construction operation been more rapidly carried out.

After about three weeks under traffic it was apparent that a seal coat was necessary. The surface was porous and raveling of the larger stone had started in numerous places. A higher percentage of fines in the mix, with a corresponding increase in cut-back asphalt, would have given a more closed surface and would probably have eliminated the necessity for a seal. The seal treatment consisted of 0.29 gallon per square yard of the same type of bituminous material as was used in the mix and a cover of 15 pounds of ⅝ to ¼ inch crushed granite spread by hand and rolled.

The cost of constructing the bituminous surface on this section was 72.58 cents per square yard or $8,516 per mile for a 20-foot width. This cost was relatively high largely because of the methods and delays in mixing and rolling.

Experiment 1, section A, stations 0+00 to 13+28.—The method of construction and the amounts of material per square yard used on this section were as follows:

Mix: 1.8 inches compacted; 154 pounds of 1¼ to ¼ inch crushed stone, 38 pounds of ¼ inch to dust, and 0.91 gallon of 60–70 penetration asphalt cut back with naphtha.

Seal: 0.42 gallon of the same bituminous material and 30 pounds of ⅝ to ¼ inch stone chips.

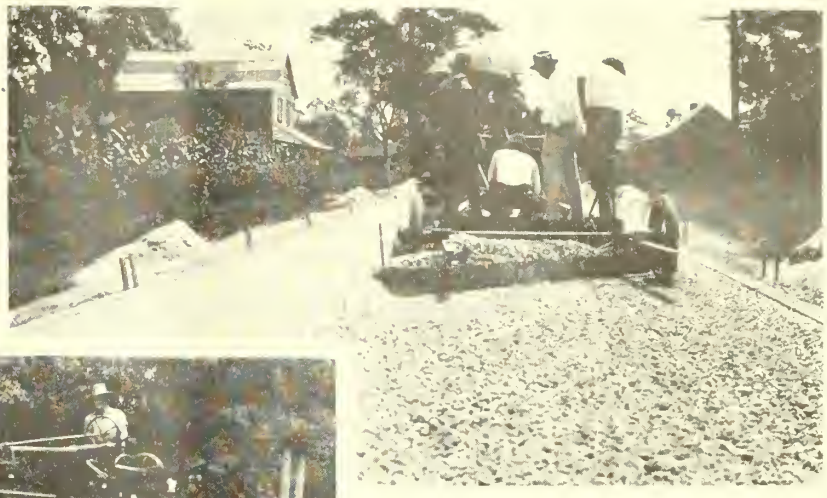
The tar primer was applied at the rate of 0.27 gallon per square yard.

The mixed surface was constructed essentially in the same manner as that of section B. The amounts of bituminous material used in the mix were 0.43 and 0.48 gallon per square yard, respectively, for the two applications. Mixing was done entirely with the blade grader. Initial rolling was done approximately 18 hours after mixing was completed and, in contrast to the previous section, a closed surface was obtained. This early rolling, having sealed a greater portion of the naphtha in the mix, necessitated rolling only in the heat of the day to prevent cracking or checking and also to obtain maximum compression. This operation was continued for a week.

The section was then opened to traffic for one month, during which period the surface showed some signs of raveling and the need of a seal treatment. This was applied in the following manner: 15 pounds of ⅝ to ¼ inch stone per square yard were spread and subjected to traffic for two days, after which the loose stone was swept to the sides and 0.42 gallon per square yard of the same material as used in the mix was applied. The stone at the sides was then respread on the surface and thoroughly rolled. After 10 days under traffic it was apparent that more cover stone was needed and an additional 15 pounds were spread but not rolled.

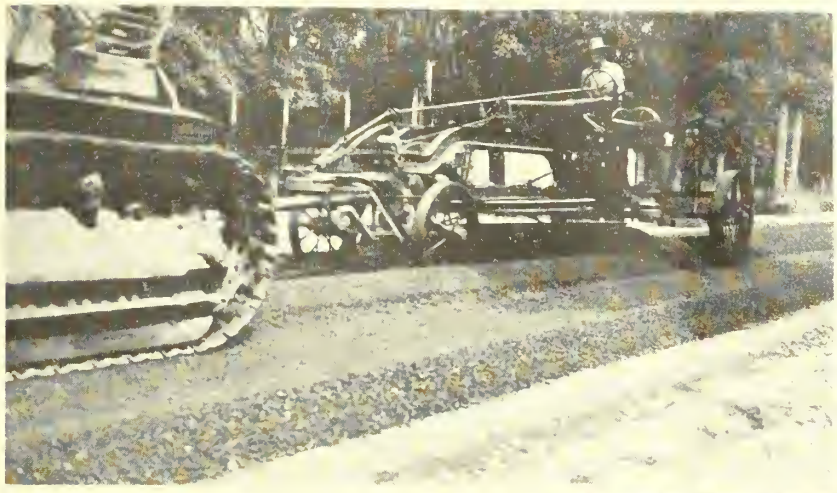
This method of first applying the stone and subjecting it to traffic prior to the application of the bituminous material was tried out experimentally with the view of developing some suitable type of seal treatment for surfaces that were in a raveled or very open condition. It proved very effective, in that the action of traffic over the loosely spread stone forced it into the larger voids, requiring less bituminous material for the seal and preventing its draining to the bottom because of the open surface texture. This method was used on some of the other sections where the surface conditions were similar to that mentioned above.

The cost of constructing this section was 66.12 cents per square yard or $7,758 per mile.

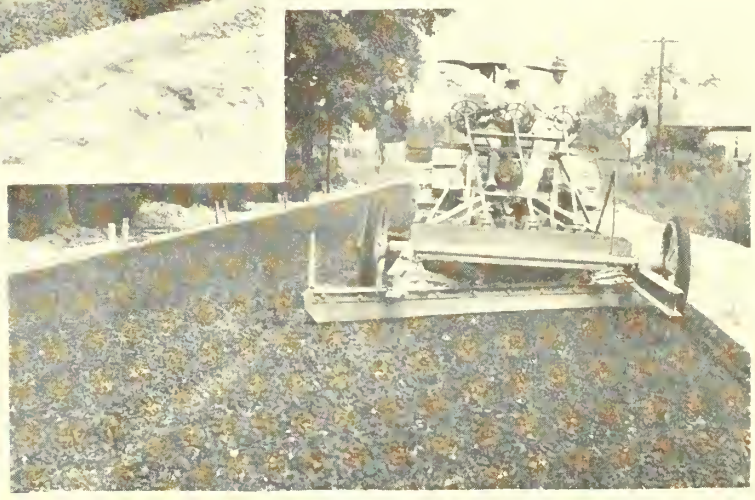


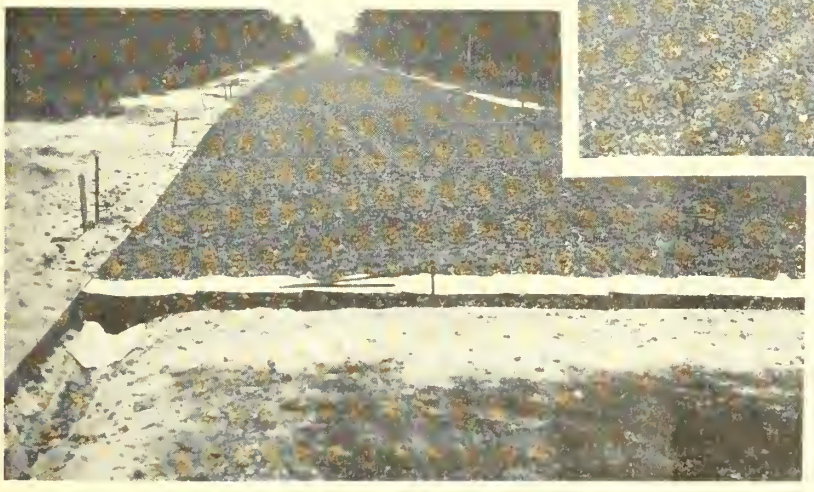
RIGHT: MIXING WITH A HEAVY FOUR-WAY DRAG DID NOT PROVE SATISFACTORY

BELOW: THE DRAG PROVED VERY SATISFACTORY FOR SMOOTHING THE SURFACE AFTER THE MIXING OPERATION WAS COMPLETED



ABOVE: MIXING WITH A BLADE GRADER WAS ENTIRELY SUCCESSFUL





LEFT: METHOD OF OBTAINING THICKENED EDGE ON THE SAND-CLAY MIXED SECTIONS BY THE USE OF BOTH FORMS AND TRENCH

FIGURE 3.—MIXED-IN-PLACE CONSTRUCTION

Experiment 2, section A, stations 24+50 to 39+60.—The method of construction and the amounts of material per square yard used on this section were as follows:

Mix: 2 inches compacted; 180 pounds of 1¼ to ¼ inch stone and 0.69 gallon of 85–100 penetration cut-back asphalt.

Seal: An average of 0.30 gallon of the same bituminous material was applied and covered with an average of 18 pounds of ⅝ to ¼ inch stone chips. This section was sealed in two separate portions.

The tar primer was applied at the rate of 0.31 gallon per square yard.

The details of construction were essentially the same as in the case of the preceding section, except that the aggregate used was 1¼ to ¼ inch in size with no fines added and the bituminous material was applied at the rate of 0.69 gallon per square yard in a single application.

The seal coat was applied three weeks later. On the portion from station 31 + 67 to 39 + 60 the same method

of seal treatment was used as was employed on experiment 1, section A; that is, the stone was spread and subjected to traffic before applying the bituminous material. Twenty-three hundredths gallon of bituminous material per square yard and about 20 pounds of ⅝ to ¼ inch stone were used on this portion. The method of seal treatment used on the remainder of the section consisted of an application of 0.38 gallon of bituminous material per square yard and covering with 15 pounds of ⅝ to ¼ inch stone.

The cost of constructing the bituminous surface of this section was 70.34 cents per square yard or $8,253 per mile.

Experiment 2, section B, stations 39+60 to 52+00.—The method of construction and the amounts of material per square yard used on this section were as follows:

Mix: 2 inches compacted; 175 pounds of 1¼ to ¼ inch crushed stone and 0.89 gallon of 25–35 viscosity tar.

Seal: 0.23 gallon of the same bituminous material and 15 pounds of ⅝ to ¼ inch stone chips.

The tar primer was applied at the rate of 0.29 gallon per square yard.

This section was constructed in essentially the same manner as the preceding section. The amount of coarse stone used was 175 pounds per square yard. An application of one-half gallon of 25–35 viscosity tar was spread on the stone but rain fell before mixing could be begun. On the following day when the materials were dried out and mixing was about completed it was evident that more bituminous material would be necessary to secure a bonded mix. Accordingly, a second application of 0.39 gallon was spread which, with additional mixing, produced a well bonded mix. The total amount in the mix was 0.89 gallon per square yard.

The seal coat was not applied until after about two months, during which traffic was permitted to use the section. No raveling occurred although the surface was fairly open. The seal consisted of an application of 0.23 gallon of the same tar as used in the mix and 15 pounds of ⅝ to ¼ inch stone. After the stone was spread the section was rolled.

The cost of constructing the bituminous surface of this section was 79.40 cents per square yard or $9,316 per mile.

Experiment 2, section C, stations 52+00 to 66+00.—The method of construction and the amounts of material per square yard used on this section were as follows:

Penetration course: 2.3 inches compacted; 165 pounds of 1¼ to ¼ inch stone, 37 pounds of ¼ inch to dust, and 1.27 gallons of asphalt emulsion.

Seal: 0.23 gallon of the same bituminous material and 15 pounds of ⅝ to ¼ inch stone chips.

The tar primer was applied at the rate of 0.28 gallon per square yard.

It was planned originally to construct this section in the same manner as the two preceding sections but the producers of the emulsion objected and requested that a modified type of penetration construction be substituted. It was decided to construct this section according to their recommendations. The construction which is here described was done under their direction and with the approval of their representatives.

The side forms were kept to line during the construction of this section. Thirty-seven pounds per square yard of stone, ¼ inch to dust, were spread upon the primed base and leveled with a strike-off board to a uniform depth of approximately ½ inch. One hundred and sixty-five pounds of 1¼ to ¼ inch stone were then spread by hand, shaped with the drag to the proper cross section, sprinkled with water, and thoroughly rolled. A strip one foot wide at each edge was given an application of emulsion, after which the entire section received an application of 0.44 gallon per square yard. Water was applied immediately to wash the emulsion down into the fine aggregate and the section was again thoroughly rolled. The purpose of the second rolling was to force the mortar of fine stone and emulsion up part way through the interstices of the coarser stone and upon drying to leave them partially embedded. Actually, however, this did not occur. The water drained away to the side ditches carrying an undetermined amount of emulsion with it. During the rolling process approximately 8 pounds of ⅝ to ¼ inch stone were spread over the surface to fill the surface voids.

FIGURE 4.—SPREADING THE COARSE STONE ON THE FINE STONE CUSHION. THIS SECTION, EXPERIMENT 2–C, WAS CONSTRUCTED WITH ASPHALT EMULSION BY A MODIFIED PENETRATION METHOD

The following day a second application of the emulsion was spread at the rate of 0.83 gallon and followed immediately with 7 pounds of ⅝ to ¼ inch stone. The section was then thoroughly rolled.

The seal coat was applied one week later after the surface had been swept and loose or raveled places patched. The treatment consisted of an application of 0.23 gallon of emulsion and 15 pounds of ⅝ to ¼ inch stone.

An inspection of the section shortly after construction showed that the first application of emulsion had not penetrated the cushion course as intended, but had to a great extent been carried off with the water used to wash it down.

The cost of constructing the bituminous surface of this section was 84.62 cents per square yard or $9,928 per mile.

Experiment 3, stations 79+20 to 92+40.—The method of construction and the amounts of material per square yard used on this section were as follows:

Mix: 1.8 inches compacted; 162 pounds of ¾ to ¼ inch stone and 38 pounds of stone ¼ inch to dust with 1.19 gallons of 85–100 penetration asphalt cut-back.

Seal: No seal treatment was required.

The tar primer was applied at the rate of 0.30 gallon per square yard.

The mixed course was constructed in the same manner as experiment 1. The coarse stone was only ¾ to ¼ inch in size but there was no tendency of the mix to ball up or segregate. A total of 1.19 gallons of bituminous material, spread in two applications, was used. After initial rolling, but while it was still in a plastic condition, the surface was somewhat rutted by local traffic which could not be detoured. A drag was used

to fill the ruts and some of the stone was thus left in a loose condition. However, it compacted under traffic and additional rolling without apparent damage to the section. The resultant mix was well closed and no seal treatment was given.

The cost of constructing the bituminous surface of this section was 61.07 cents per square yard or $7,165 per mile.

Experiment 4, stations 92+40 to 105+84. The method of construction and the amounts of material per square yard used on this section were as follows:

Mix: 1.8 inches compacted; 157 pounds of ¾ to ¼ inch stone and 0.72 gallon of 85–100 penetration asphalt cut-back.

Seal: An average of 0.22 gallon of the same bituminous material was applied and covered with 15 pounds of ⅝ to ¼ inch stone chips. This section was sealed in two separate portions.

The tar primer was applied at the rate of 0.26 gallon per square yard.

The method of construction of this section was essentially the same as that of sections 2–A and 2–B except that the coarse stone was ¾ to ¼ inch. Seventy-two hundredths gallon of asphalt was spread in one application which apparently was not enough, as the mix did not compact and knead to any great extent under the roller. Upon opening to traffic the surface raveled but the approach of cold weather made remixing undesirable and it was decided to correct surface failures by means of a seal treatment. Two adjacent portions of the section, each about 100 feet long, were selected for trying out, in an experimental way, methods of treating the section. On one portion the loose stone was swept to the roadside and piled, while on the other it was uniformly redistributed on the surface. Both portions were then given an application of approximately one-third gallon of bituminous material per square yard. The stone was respread on the first portion with an additional amount to make about 15 pounds per square yard and both portions then thoroughly rolled. Rolling of the portion on which the stone was not removed and respread presented little difficulty. When the bituminous material became somewhat sticky, the use of kerosene on the roller wheels prevented picking up. It was noted on the following day that the bituminous material had penetrated the mix to approximately the same depth on both portions and that the area on which the loose stone was first removed showed a somewhat better surface appearance. Consequently this method was used to seal the remaining portion. Twenty-two hundredths gallon of bituminous material was used with a total of 15 pounds of stone.

The cost of constructing the bituminous surface of this section was 55.46 cents per square yard or $6,507 per mile.

SECTION GIVEN ORDINARY SURFACE TREATMENT

Experiment 5, stations 66+00 to 79+20.—The method of construction and the amounts of material per square yard used on this section were as follows:

Forty-four hundredths gallon of 85–100 penetration asphalt cut-back was applied and covered with 50 pounds of 1¼ to ¼ inch crushed stone.

Retreatment: (Stations 66+00 to 73+40).

Thirty-six hundredths gallon of the same bituminous material was applied and covered with 15 pounds of ⅝ to ¼ inch crushed stone.

The tar primer was applied at the rate of 0.33 gallon per square yard. The asphalt was applied at the rate of 0.44 gallon per square yard and immediately covered with 50 pounds of 1¼ to ¼ inch stone. The surface was then throughly rolled but was not opened to traffic until the following day. The rolling operation was continued intermittently for several days, during which the stone displaced by traffic was respread with brooms and shovels. Some early patching was required and the portion between stations 66+00 and 73+40 was sealed three months after the construction. For the seal coat, 0.36 gallon of the 85–100 penetration cut-back asphalt and 15 pounds of ⅝ to ¼ inch stone were spread and rolled.

The cost of constructing this surface treatment was 28.47 cents per square yard or $3,340 per mile.

EXPERIMENTAL SECTIONS CONSTRUCTED ON SAND-CLAY BASE

The experimental sections built on the sand-clay base were identical in type of construction and material used with those constructed on the marl base and are numbered correspondingly; i. e., Nos. 1 and 6 are identical, 2 and 7, etc.

Prior to applying the primer on the mixed sections, a triangular trench was cut with the grader on each side of the road to provide for a thickening of the edges. The trench was approximately 1 foot wide and 4 inches deep on the outside edge but was very irregular because of the nonuniformity of the sand-clay. The sections were swept and all loose material was removed, following which action the base was primed with 8–13 viscosity tar applied at a temperature of 125° to 155° F. As was expected, the penetration into the sand-clay base varied with the composition of the material penetrated. In general, the primer penetrated readily and appeared to be of considerable assistance in bonding the surface and forming a stable crust. During the mixing process the primed base was damaged considerably in some areas by the wheels of the road machines and the cleats of the tractors.

Side forms were used on some sections as noted. Edge protection was provided after construction by building up the shoulders with sand-clay.

MIXED-IN-PLACE SECTIONS

Experiment 6, section A, stations 105+84 to 119+99.—This section corresponds to Experiment 1, section A. The method of construction and the amounts of material used per square yard were as follows:

Mix: 2 inches compacted; 171 pounds of 1¼ to ¼ inch stone, 40 pounds of ¼ inch to dust, and 1 gallon of 60–70 penetration asphalt cut back with naphtha.

Seal: 0.34 gallon of 85–100 penetration asphalt cut-back was applied and covered with 15 pounds of ⅝ to ¼ inch stone chips.

The tar primer was applied at the rate of 0.33 gallon per square yard.

After the surface, including the area occupied by the trenches, was primed the side forms were placed inside the trenches to prevent the untreated stone from getting into them. The process of mixing was the same as that used on the corresponding section on the marl base. The bituminous material was applied in two applications at the rates of 0.41 and 0.59 gallon, respectively. The appearance and behavior of this mix was very similar to that of the corresponding section on the marl base, in that its surface appeared lean and raveled somewhat immediately following construction. A seal coat was deemed necessary and was applied one month after construction. It consisted of 0.34 gallon

of 85–100 penetration asphalt cut-back and a cover of 15 pounds of ⅝ to ¼ inch stone which was spread by hand and rolled. Substitution of the 85–100 cut-back for the 60–70 material was done through error.

The cost of constructing the bituminous surface of this section was 72.53 cents per square yard or $8,510 per mile.

Experiment 6, section B, stations 119+99 to 132+00.—This section corresponds to experiment 1, section B. The method of construction and the amounts of material used per square yard were as follows:

Mix: 2 inches compacted; 170 pounds of 1¼ to ¼ inch crushed stone, 42 pounds of ¼ inch to dust and 1.09 gallons 85 100 penetration asphalt cut back with naphtha.

Seal: 0.23 gallon of the same bituminous material and 15 pounds of ⅝ to ¼ inch stone chips.

The tar primer was applied at the rate of 0.39 gallon per square yard. In general, this quantity penetrated readily. However, on a portion of the section situated on a superelevated curve the amount was somewhat excessive as some of the material flowed to the side trenches.

An application of asphalt, 0.49 gallon, was made on the coarse aggregate late in the day and immediately covered with fines. The following morning the second application, 0.60 gallon, was spread and mixing was begun. During the mixing operation the primed surface of the base was damaged in a few places by the tractors and road machines. The sand-clay thus loosened was removed as much as possible. After mixing and spreading were completed but before the section was rolled a heavy rain fell, which penetrated the mix to the base. During the rolling process a number of spongy areas appeared where the softened base had worked into the mix. These were removed and replaced with good material.

Traffic was permitted on the section for about three months before it was sealed. During this time very little raveling had occurred although the surface appeared fairly lean.

The seal coat consisted of an application of 0.23 gallon of the same bituminous material as was used in the mix, covered with 15 pounds of ⅝ to ¼ inch stone and rolled.

The cost of constructing the bituminous surface of this section was 72.10 cents per square yard or $8,459 per mile.

Experiment 7, section A, stations 132+00 to 144+20.—This section corresponds to experiment 2, section A. The method of construction and the amounts of material used per square yard were as follows:

Mix: 2 inches compacted; 178 pounds of 1¼ to ¼ inch stone and 0.77 gallon of 85–100 penetration asphalt cut-back. Mixed, shaped, and rolled.

Seal: 0.27 gallon of the same bituminous material was applied and covered with 18.3 pounds of ⅝ to ¼ inch stone chips.

The tar primer was applied at the rate of 0.29 gallon per square yard. The penetration averaged ⅜ inch. Two days after priming the surface was dry and hard. The side forms were not used during the construction of this section.

The bituminous material was applied at the rate of 0.77 gallon in two applications. As in the case of experiment 6–B the primed base was damaged to some extent during the mixing. This section also was subjected to rain during mixing, the fairly open mix permitting the moisture to enter and penetrate to the base. The mix was already rather stiff and the base was softened so that additional mixing was not deemed advisable after the rain. When rolled the surface appeared in good condition except for considerable moisture which was retained in the mix. Traffic was not permitted on the section until it had dried out.

One week later the seal treatment was applied. The small stone was first spread and allowed to lie under traffic for a few days, after which the loose stones were broomed to the sides and 0.27 gallon of the same bituminous material as used in the mix was applied. The stone was then respread with an additional amount to make a total of about 18 pounds. Rolling of the surface completed the construction.

A few days later inspection of the section showed the base to be dry and the surface well sealed.

The cost of constructing the bituminous surface of this section was 62.86 cents per square yard or $7,375 per mile.

Experiment 7, section B, stations 199+70 to 212+90.—This section corresponds to experiment 2, section B. The method of construction and the amounts of material used per square yard were as follows:

Mix: Two inches compacted; 169 pounds of 1¼ to ¼ inch crushed stone and 0.77 gallon of 25–35 viscosity tar.

Retreatment: Stations 206+50 to 212+90, seal coat applied, using 0.42 gallon of the same bituminous material and 15 pounds of ⅝ to ¼ inch stone chips. Station 199+70 to 206+50, remixed with 0.30 gallon of the same bituminous material. Sealed with 0.23 gallon of same bituminous material and 15 pounds of ⅝ to ¼ inch stone chips.

The tar primer was applied at the rate of 0.25 gallon and the penetration averaged ⅜ inch.

The side forms were not used during the construction of this section. Seventy-seven hundredths gallon of tar was applied and the mixing operation carried out as in the preceding section. When the section was completed it was evident that an insufficient quantity of tar had been used in the mix, as it was not properly bonded and considerable raveling took place under traffic. Because of the cool weather remixing was not believed advisable and it was decided to apply a rather heavy seal to make up for the deficiency of tar in the mix. A tar seal was applied on the north portion (station 206+50 to 212+90) a few days after construction, at the rate of 0.42 gallon, and covered with 15 pounds of ⅝ to ¼ inch stone chips per square yard. The tar supply ran out at this point and the additional amount necessary to complete the job was not received for about 10 days. During this period the remainder of the section had become so raveled and displaced under traffic that remixing was considered necessary. This portion was first scarified or torn up with the blade grader and given an application of 0.30 gallon of tar per square yard. It was then remixed, spread, and rolled.

Approximately two months later it was necessary to give the portion of the section that had been remixed a light seal treatment in order to close the surface to prevent the entrance of moisture. Prior to applying the seal, 15 pounds per square yard of ⅝ to ¼ inch stone were spread on the surface, allowing traffic to force some of the particles into the mix. After two days the loose stone was swept to the sides and 0.23 gallon of tar applied. The stone was then respread over the surface and rolled.

The cost of constructing the bituminous surface of the section was 73.23 cents per square yard or $8,592 per mile.

Experiment 7, section C, stations 144+20 to 158+40.—This section corresponds to experiment 2, section C.

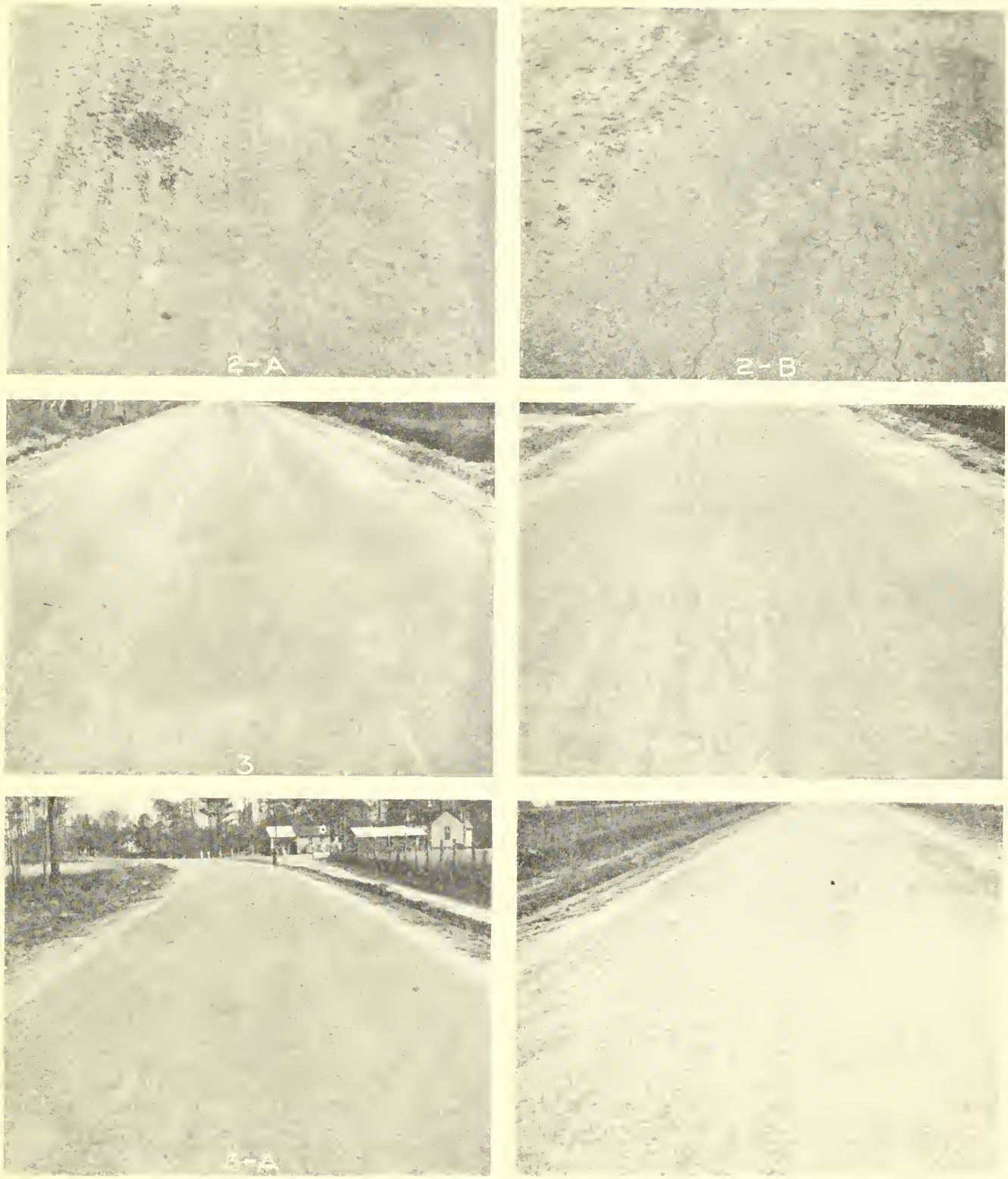


FIGURE 5.—EXAMPLES OF SURFACE APPEARANCE SIX MONTHS AFTER CONSTRUCTION. THE NUMBERS OF THE EXPERIMENTAL SECTIONS ARE GIVEN ON THE PHOTOGRAPHS. EXPERIMENTS 2-A AND 2-B: CRACKING AND RAVELING DURING SPRING OF 1930 WAS CAUSED BY SOFTENING OF THE MARL BASE AS A RESULT OF MOISTURE. EXPERIMENT 3: SECTION IN EXCELLENT CONDITION; MARL BASE, WHILE SLIGHTLY DAMP, HAD NOT SOFTENED AS IN EXPERIMENTS 2-A AND 2-B. EXPERIMENT 4: SURFACE IN GOOD CONDITION, EXCEPT FOR A FEW SMALL AREAS WHERE MARL BASE HAS SOFTENED. EXPERIMENT 6-A: IN EXCELLENT CONDITION; SAND-CLAY BASE MOIST, BUT ITS SUPPORTING VALUE APPARENTLY NOT AFFECTED. EXPERIMENT 10: GOOD CONDITION DUE TO EARLY RE-TREATMENT, WHICH NO DOUBT SAVED THE SECTION FROM COMPLETE DISINTEGRATION

The method of construction and the amounts of material used per square yard were as follows:

Penetration course: 2.3 inches compacted; 165 pounds of 1¼ to ¼ inch stone, 30 pounds of ¼ inch to dust and 1.11 gallons of asphalt emulsion.

Seal: 0.25 gallon of the same bituminous material and 15 pounds of ⅝ to ¼ inch stone chips.

The sand-clay base on this section was generally better than the average, as were the drainage conditions.

The tar primer was applied at the rate of 0.34 gallon and gave an average penetration of ⅜ inch.

Side forms were not used on this section. Otherwise the details of construction were essentially the same as those of the corresponding section on the marl base. As no mixing was done on this section the primed base was not disturbed during the construction of the bituminous mat. The emulsion was applied at the rate of 0.32 gallon per square yard for the first application and 0.79 gallon for the second. As on experiment 2, section C, where similar materials and methods were used, an appreciable amount of the first application was carried off by the water used to wash it down into the fine stone cushion. After rolling, the section was comparatively rough and, while under traffic for about 10 days, raveled considerably. Before the seal treatment was applied depressions and surface failures were repaired with emulsion and ½-inch stone. The surface was then swept and 0.25 gallon of emulsion applied and covered with 15 pounds of ⅝ to ¼ inch stone.

As on the other emulsion section, it was found that the emulsion had not penetrated the fine stone cushion.

The cost of constructing the bituminous surface of this section was 84.55 cents per square yard or $9,920 per mile.

Experiment 8, stations 212+90 to 226+10.—This section corresponds to experiment 3. The method of construction and the amounts of material used per square yard were as follows:

Mix: 1.8 inches compacted; 154 pounds of ¾ to ¼ inch stone, 38 pounds ¼ inch to dust, and 1.25 gallons of 85–100 penetration asphalt cut-back.

Seal: No seal treatment was required.

The sand-clay base on this section was very nonuniform and during construction compacted under traffic into strata which resulted in some scaling, especially near the edges. All loose material was removed before priming. The prime coat was applied at the rate of 0.26 gallon per square yard. The section was constructed similarly to the corresponding section on the marl base except that side forms were not used. The asphalt was spread in two applications at the rate of 0.51 and 0.74 gallon per square yard, respectively. Before mixing was completed a heavy rain occurred but the mix was windrowed in the center and was therefore partially protected. Mixing was not resumed until the base was thoroughly dry. After the mix was shaped and partially rolled, but before it hardened, it was badly rutted by local traffic on the following day. On the next day the surface was loosened with a disk harrow, reshaped, and again rolled. Although the mix had become somewhat stiff no difficulty was encountered in obtaining a well compacted, close surface. A seal was not considered necessary.

The cost of constructing the bituminous surface of the section was 59.53 cents per square yard or $6,985 per mile.

Experiment 9, stations 226+10 to 238+80.—This section corresponds with experiment 4. The method of construction and the amounts of material used per square yard were as follows:

Mix: 1.8 inches compacted; 152 pounds of ¾ to ¼ inch stone and 0.77 gallon 85–100 penetration cut-back.

Seal: 0.29 gallon of the same bituminous material was applied and covered with 14 pounds of ⅝ to ¼ inch stone chips.

One month preceding the construction of this section the sand-clay base was plowed and scarified, and additional clay binder was added. It was then machined and opened to traffic for compaction. At the time the surface was constructed it was in fair condition.

The tar primer was applied at the rate of 0.27 gallon and the penetration averaged about ⅜ inch. Side forms were not used.

The bituminous material was applied at the rate of 0.77 gallon per square yard. In the 1½ months following construction, during which time the section was under traffic before sealing, the surface raveled considerably, probably because of leanness as well as the lack of fine material necessary to give a well-graded mixture.

Before the application of the seal coat, the loose stone was swept to the sides of the road. The bituminous material was then applied at the rate of 0.29 gallon per square yard, and the stone respread with an additional amount to make a total of 14 pounds per square yard. Rolling completed the treatment.

The cost of constructing the bituminous surface of this section was 66.74 cents per square yard or $7,831 per mile.

SECTION GIVEN ORDINARY SURFACE TREATMENT

Experiment 10, stations 158+40 to 199+70 (less 170 feet).—This section corresponds to experiment 5. The method of construction and the amounts of material used per square yard were as follows:

Forty-six hundredths gallon of 85–100 penetration asphalt cut back with naphtha and 52 pounds of 1¼ to ¼ inch crushed stone.

Retreatment: 0.32 gallon of the same bituminous material covered with the loose stone that had been whipped off by traffic, plus 13 pounds of ¾ to ¼ inch crushed stone.

The sand-clay base on this section was in only fair condition. At the time of construction it had compacted under traffic in layers and, in many widely distributed areas, some peeling occurred during sweeping, preparatory to application of the primer.

Four days before the prime coat was applied the sand-clay surface was sprinkled with water and lightly machined. The tar primer was applied at the rate of 0.29 gallon per square yard. The primed surface was in good condition when surface treated except for a few local areas where traffic had caused some flaking of the laminated sand-clay. Before treatment, such loosened material was removed and the uncovered areas painted with 85–100 penetration asphalt cut-back. The surface treatment consisted of applying 0.46 gallon of the asphalt and 52 pounds of 1¼ to ¼ inch stone per square yard. The section was then thoroughly rolled. During rolling considerable failure of the sand-clay base occurred. On many widely scattered areas the upper portion of the base, which before treatment had appeared stable, separated from the lower portion and worked up through the cover stone. Under traffic these areas quickly disintegrated and whipped off leaving the base exposed.

As the number of areas failing in this manner was so numerous that it was deemed impractical to patch each one, a portion of the section was selected for experi-

mentation to devise a satisfactory method of correcting the failure. The portion so selected was between stations 179+20 and 184+15, which was probably in the worst condition of any of the section. The loose stone, estimated at about 25 pounds per square yard, was swept to the sides and an application of 0.32 gallon of the asphalt cut-back applied, after which the stone was respread with 10 pounds additional. This treatment proved satisfactory and the remainder of the section was treated in the same manner, except that the new stone added was ¾ to ¼ inch, spread at the rate of 13 pounds per square yard, and the amount of bituminous material used was 0.31 gallon per square yard.

The cost of constructing this surface treatment, including the re-treatment, was 29.44 cents per square yard or $3,454 per mile.

DISCUSSION OF RESULTS OBTAINED

SEAL COAT NECESSARY WITH COARSER AGGREGATES

The open surface obtained on experiments 1 and 6, sections A and B, on which 1¼ to ¼ inch stone was combined with the stone screenings graded from ¼ inch down, indicated that the 20 per cent of fine aggregate used was not sufficient to close the voids in the large size stone and that a seal coat was necessary. On experiments 3 and 8, where the ¾ to ¼ inch stone was combined with the fine aggregate in the same proportion, a close mix was obtained which did not require a seal treatment. For the coarser stone it is believed that at least 30 per cent of fines would have been necessary in order to produce a surface similar to that obtained with the ¾ inch to dust.

On experiments 2 and 7, sections A and B, on which 1¼ to ¼ inch stone was used and on experiments 4 and 9 on which ¾ to ¼ inch stone was used without the addition of fines, the original plan was to close the surface with a key course, using ½ to ¼ inch stone. This was found to be impractical as the surfaces were not sufficiently open and the addition of stone of this size would not have filled the surface voids but would merely have added thickness to the mat. It was decided that, for this type of construction and size of aggregate, a better method would be to omit the key course and use a fine aggregate as a seal treatment in order to insure a closed surface. This type of treatment was therefore used on these sections.

MIXING METHODS DISCUSSED

In the original design no mention was made of the method of mixing. It was thought that in those sections where the fines were to be added to the coarse aggregate some difficulty might be encountered because of segregation, particularly if a drag were used in the mixing. To prevent this it was decided to treat the coarse stone with a light application of bituminous material before adding the fines. Although this method of applying the fines was thought necessary for the drag mixing, it would have not been necessary where mixing was done with a blade machine. For this method of construction a crusher-run stone could probably have been used equally as well and at less cost.

The cut-back asphalt used in this work was manufactured under the same specification as that used for surface treatment work and mixed-in-place construction where sand-clay and topsoil material are used as aggregates. Although no construction difficulty was encountered in the use of this material, there are reasons to believe that had it been of higher viscosity it would have proved more satisfactory in that a greater film thickness of bitumen on the stone would have been obtained, a condition which is probably desirable with the coarser aggregates. Another advantage of using the higher viscosity is that less distillate would be required, resulting in a somewhat cheaper material. The viscosity of the material used on this work ranged from about 23 to 37, Engler, at 50° C. From experience in other States on similar work, it is believed that a material of at least 50 viscosity could have been used.

In building a surface mat by the mixed-in-place method it is very important, particularly when using asphaltic materials of the cut-back type, to have the mix in a condition of tackiness before compacting it either by traffic or by rolling. This is best accomplished by spreading and shaping the mix immediately after mixing and allowing sufficient time to lapse before rolling in order that the light distillates may evaporate. Another method commonly used is that of windrowing the material immediately after mixing and leaving it to cure before spreading and rolling. This latter method requires a longer curing period, is likely to cause segregation at the time of spreading and, in general, has not proved as satisfactory as the first method, as well as being somewhat more costly. There is also a possibility of some of the bituminous material draining to the bottom of the windrow, causing the mix to be nonuniform and resulting in fat and lean streaks in the surface after the roadway is opened to traffic.

Experience has shown that a smoother and more stable surface can be obtained in the early life of the pavement by either of these two plans than can be obtained when the mix is laid and rolled before the proper tackiness has developed. Rolling the mix before an appreciable amount of the light distillate has volatilized results in this material being trapped in the mix, causing it to remain unstable for some time. It is believed that on a considerable amount of mixed-in-place construction with cut-back asphalt, compacting the mix immediately after mixing without sufficient time for curing has resulted in early shoving and often in unsatisfactory surface behavior of the mat.

The ⅝ to ¼ inch stone used as cover material was not altogether satisfactory. A smaller size stone would have proved more satisfactory from the standpoint of waterproofing the surface. However, the larger size cover resulted in added thickness and in the formation of a somewhat rough mat with some of the larger stone projecting, rendering the pavement more skid proof.

ALTERNATIVE METHODS OF SURFACE TREATMENT DESCRIBED

In view of the difficulty experienced during construction of the surface treatment on the sand-clay because of the scaly, raveled condition, it is probable that a better method of applying a surface treatment, particularly for this base condition, would be to apply the heavy binding material in two applications so that the second application would heal any breaks in the first. The treatment might consist of spreading 0.2 gallon of binder over the prime coat and covering with stone, rolling, and then spreading the remaining 0.3 gallon, followed by a covering of key stone and rolling. A seal coat treatment could be applied later as required.

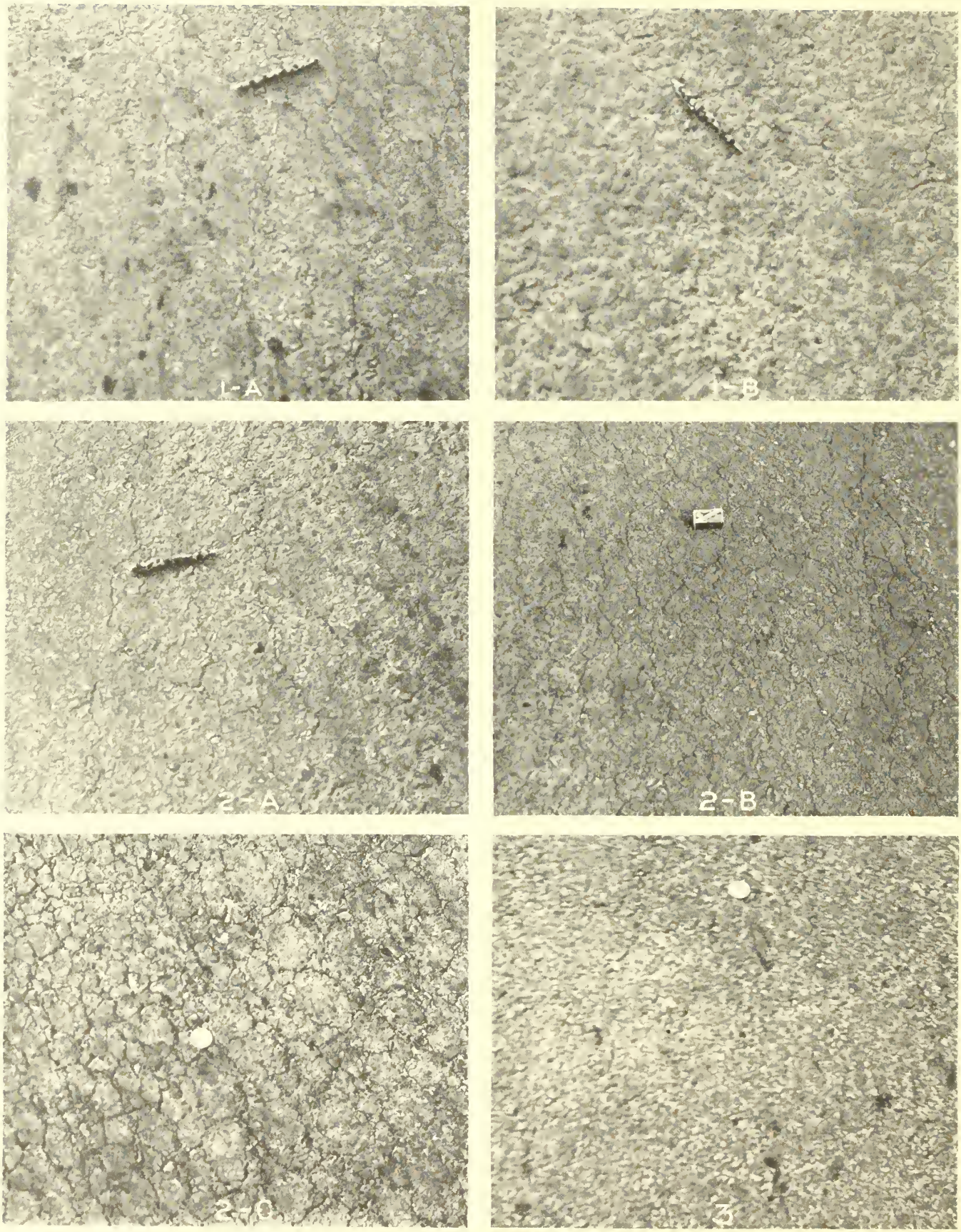


FIGURE 6.—TYPICAL EXAMPLES OF SURFACE CONDITION DURING THE SPRING OF 1931. THE NUMBERS OF THE EXPERIMENTAL SECTIONS ARE GIVEN ON THE PHOTOGRAPHS

TABLE 9.—*Cost of construction*

[Cost in cents per square yard]

Experiment number	1		2			3	4	5	6		7			8	9	10
Section	A	B	A	B	C				A	B	A	B	C			
Area, square yards	2,951	2,493	3,275	2,755	3,111	2,933	2,987	2,933	3,144	2,669	2,711	2,933	3,156	2,933	2,822	8,800
Type of base	Marl								Sand-clay							
Type of construction	Mix	Mix	Mix	Mix	Penetration	Mix	Mix	Surface treatment	Mix	Mix	Mix	Mix	Penetration	Mix	Mix	Surface treatment
Bituminous materials used	60–70 cut-back	85–100 cut-back	85–100 cut-back	25–35 tar	Asphalt emulsion	85–100 penetration cut-back			60–70 cut-back	85–100 cut-back	85–100 cut-back	25–35 tar	Asphalt emulsion	85–100 penetration cut-back		
ITEMS OF CONSTRUCTION																
Preparation of base	0.31	1.28	1.36	0.40	0.39	0.45	0.62	1.60	1.60	2.70	0.89	2.76	0.84	1.64	12.49	1.24
Prime treatment in place	3.65	3.63	4.10	3.88	3.69	4.01	3.54	4.48	4.39	5.16	3.92	3.35	4.58	3.41	3.62	3.92
Mixed material:																
Aggregate in place	35.05	36.77	32.66	33.50	37.34	34.96	29.96		38.00	38.52	34.10	31.64	40.07	34.95	29.46	
Bituminous material in place	7.96	7.69	5.13	14.72	21.31	8.90	5.36		8.74	8.13	5.74	12.69	18.73	9.32	5.76	
Mixing and shaping	2.00	5.28	3.30	7.86	1.42	2.78	2.55		3.34	3.27	3.12	5.29	.65	2.32	1.95	
Rolling	1.49	2.50	3.12	1.42	4.85	1.40	2.00		1.30	2.10	2.69	2.27	4.69	1.15	1.78	
Total cost of mixed material	46.50	52.24	44.21	57.50	64.92	48.04	39.87		51.38	52.02	45.65	51.89	64.14	47.74	38.95	
Surface treatment:																
Aggregate in place								10.12								12.45
Bituminous material in place								3.28								5.84
Rolling								.50								.53
Total cost of surface treatment								13.90								18.82
Seal treatment:																
Bituminous material in place	3.66	2.17	3.51	3.80	3.80		1.69	1.52	2.50	1.67	1.98	5.36	4.18		2.13	
Aggregate in place	2.89	2.89	2.23	2.74	2.89		2.90	1.39	2.89	2.89	3.52	2.89	2.89		2.89	
Rolling	.93	1.20			1.09			.07	.20	.14			.87			
Total cost of seal treatment	7.48	6.26	5.74	6.54	7.78		4.59	2.98	5.59	4.70	5.50	8.25	7.94		5.02	
Incidentals:																
Forms	.05	.78	.07	.22	.32	.95	.15		2.04	.16						
Shoulders	1.82	1.91	8.44	4.21	.74	1.44	.65	.16	1.05	.90	.67	.49	.27	.60	.33	.08
Demurrage	.75	.75	.75	.75	.75	.75	.75	.75	.75	.75	.75	.75	.75	.75	.75	.75
Engineering	1.68	1.85	1.79	2.02	2.15	1.55	1.41	.72	1.85	1.83	1.60	1.86	2.15	1.51	1.70	.75
Miscellaneous	3.88	3.88	3.88	3.88	3.88	3.88	3.88	3.88	3.88	3.88	3.88	3.88	3.88	3.88	3.88	3.88
Total cost of incidentals	8.18	9.17	14.93	11.08	7.84	8.57	6.84	5.51	9.57	7.52	6.90	6.98	7.05	6.74	6.66	5.46
Total cost per square yard	66.12	72.58	70.34	79.40	84.62	61.07	55.46	28.47	72.53	72.10	62.86	73.23	84.55	59.53	66.74	29.44
Cost per mile, 20-foot width	$7,758	$8,516	$8,253	$9,316	$9,928	$7,165	$6,507	$3,340	$8,510	$8,459	$7,375	$8,592	$9,920	$6,985	$7,831	$3,454

Another method of meeting the difficulty of loose subgrade in the case of light surface treatment would be to treat the surface of the base. For example, a heavier prime coat might be used, followed by light surface mixing and rolling if necessary. It is probably often the case that the soil can be so treated or the treatment of soil roads can be so modified that light bituminous wearing surfaces can be constructed without expensive rebuilding of the subgrade.

Because of the experimental character of the sections and their relatively short lengths, the cost as shown in Table 9 naturally was higher than that of constructing any one section of a considerable length. The type of construction used on these experiments offers an opportunity for the use of mechanical equipment, thereby reducing hand labor to a minimum with a corresponding reduction in cost. As indicated in the description of the various sections, considerable hand labor was employed.

MAINTENANCE AND EARLY BEHAVIOR

A record of the maintenance costs on the different sections of this project are given in Table 10. Inspection of the project has been made at various intervals and the comparative behavior is shown by the illustrations accompanying this article and in the following discussion.

During the spring following the construction all the sections on the marl base, excepting experiment No. 5, the plain surface treatment, experiment No. 3, ¾ to ¼ inch stone plus fines, and experiment No. 4, ¾ to ¼ inch stone without the fines, had developed extensive cracking and presented, to a considerable extent, an "alligator-hide" appearance. This condition was found to be due to the softening of the marl base caused by capillary as well as surface moisture. The presence of the prime coat on the marl did not seem to prevent the upward movement of moisture through the base. The upper surface of the marl which had been penetrated by the primer was also softened and contained a high percentage of moisture.

All of the experiments on the sand-clay were found to be in excellent condition. Examination of this base showed that it contained as much moisture as the marl, and sometimes more, but the primed surface had not been affected by the moisture. The stable crust formed by the primer had to a large extent prevented the upward movement of the base moisture into the bituminous wearing surface.

A later inspection, made during the summer of 1930, showed that both the marl and the sand-clay bases had to a great extent dried out and that the marl had hardened considerably. It was thought possible that, as the base support of the sections constructed

TABLE 10.—*Cost of maintenance from October, 1929, to December, 1930*

Experiment No.	Section No.	Type of base	Original construction August–September, 1929					Maintenance costs		
			Bituminous material	Type of surface	Size of aggregate	Cost per square yard	Cost per mile 20 feet wide	October–December, 1929—Per square yard	January–December, 1930—Per square yard	Per mile per year
					Inches	*Cents*		*Cents*	*Cents*	
1	A	Marl	Asphalt, cut-back	Mixed	1¼-dust	66.12	$7,758	0.00	0.82	$77
1	B	do	do	do	do	72.58	8,516	.12	1.48	150
2	A	do	do	do	1¼–¼	70.34	8,253	.05	1.10	108
2	B	do	Tar	do	do	79.40	9,316	.10	1.74	173
2	C	do	Asphalt emulsion	Penetration	1¼-dust	84.62	9,928	.00	4.21	395
3		do	Asphalt cut-back	Mixed	¾–dust	61.07	7,165	.15	.23	37
4		do	do	do	¾–¼	55.46	6,507	.00	1.64	154
5		do	do	Surface treatment	1¼–¼	28.47	3,340	.10	4.29	413
Average [1]										177
6	A	Sandclay	do	Mixed	1¼-dust	72.53	8,510	.08	.44	49
6	B	do	do	do	do	72.10	8,459	.11	1.09	113
7	A	do	do	do	1¼–¼	62.86	7,375	.10	1.31	132
7	B	do	Tar	do	do	73.23	8,592	.00	1.18	111
7	C	do	Asphalt emulsion	Penetration	1¼-dust	84.55	9,920	.19	1.05	117
8		do	Asphalt cut-back	Mixed	¾-dust	59.53	6,985	.09	.33	39
9		do	do	do	¾–¼	66.74	7,831	.00	.92	86
10		do	do	Surface treatment	1¼–¼	29.44	3,454	.22	1.69	179
Average [1]										111

[1] Average weighted for lengths of experimental sections.

on the marl base increased, the cracks which had developed during the winter would heal under the summer traffic. During the fall it was evident that little or no healing had occurred, although no new cracks had developed during the summer.

During the following winter of 1930–31 a repetition of the same conditions occurred on the marl base and an inspection of the project made during the spring of 1931 showed that the cracking had increased, the old cracks had become wider and, in some few areas, raveling had started to a limited extent. The surface of the three experiments on the marl base that were not affected during the first winter were still in good condition and comparable with those on the sand-clay base.

A comparison of the various experiments, based on their past behavior and their condition in the spring of 1931, shows that those constructed on the sand-clay base are far better than those on the marl, except for experiments Nos. 5, 3, and 4, which compare favorably with those on the sand-clay base. A rating of the experiments based on their present surface condition is given in Table 11.

The surface maintenance of the various experimental sections has consisted of hand patching in certain local areas where failures have occurred. Prior to the summer of 1931 no re-treatments had been made on any of the experiments. The data given in Table 10 show the average maintenance costs for the experiments on the marl base to be $177 per mile per year while the maintenance cost for those on the sand-clay base has been $111 per mile per year. Although these costs indicate a distinct superiority on the part of the sand-clay base, the true merits of the two types of base are not fully shown by these figures, as the experiments constructed on the sand-clay will require no great amount of maintenance in the near future while the experiments constructed on the marl base, with the exception of experiments 3, 4, and 5 will probably require some type of re-treatment.

Two types of re-treatment of these surfaces were considered and were tried out experimentally during the summer of 1931. Under one of these methods the present surface is scarified and remixed with additional bituminous material, after which the mixture is again spread and rolled. Under the other method a light surface treatment is applied to the surface of the existing experiment. From the early results of these two types of treatments the better method will be selected for re-treating the remainder of the experiments on the marl base which are in need of re-treatment.

The unsatisfactory condition of the majority of the experiments constructed on the marl base has been caused by the softening of this material during the wet winter months. Evidently this material, the analysis of which is given on page 217, is not suitable for base construction where the moisture conditions are as unfavorable as they are in the locality in which this project was constructed. It is possible that with a higher elevation of the road surface and deeper drainage ditches, which would no doubt reduce the capillary moisture, the be-

TABLE 11.—*Relative ratings of experimental sections*

MIXED-IN-PLACE ON SAND-CLAY BASE

1—Experiment 8, ¾ to ¼ inch stone plus fines, 85–100 penetration asphalt cut-back.
2—Experiment 6, Section B, 1¼ to ¼ inch stone plus fines, 85–100 penetration asphalt cut-back.
3—Experiment 6, section A, 1¼ to ¼ inch stone plus fines, 60–70 penetration asphalt cut-back.
4—Experiment 9, section A, ¾ to ¼ inch stone, no fines, 85–100 penetration asphalt cut-back.
5—Experiment 7, section B, 1¼ to ¼ inch stone, no fines, 25–35 viscosity tar.
5—Experiment 7, section A, 1¼ to ¼ inch stone, no fines, 85–100 penetration apshalt cut-back.
6—Experiment 7, section C, 1¼ to ¼ inch stone plus fines, asphalt emulsion.

MIXED-IN-PLACE ON MARL BASE

1—Experiment 3, same as experiment 8.
2—Experiment 4, same as experiment 9.
3—Experiment 1, section A, same as experiment 6, section A.
3—Experiment 1, section B, same as experiment 6, section B.
4—Experiment 2, section A, same as experiment 7, section A.
4—Experiment 2, section B, same as experiment 7, section B.
5—Experiment 2, section C, same as experiment 7, section C.

SURFACE TREATMENT

Experiment No. 5, on marl base, and experiment No. 10, on sand-clay base, were both in excellent condition. There was no noticeable difference in the condition of the two experiments.

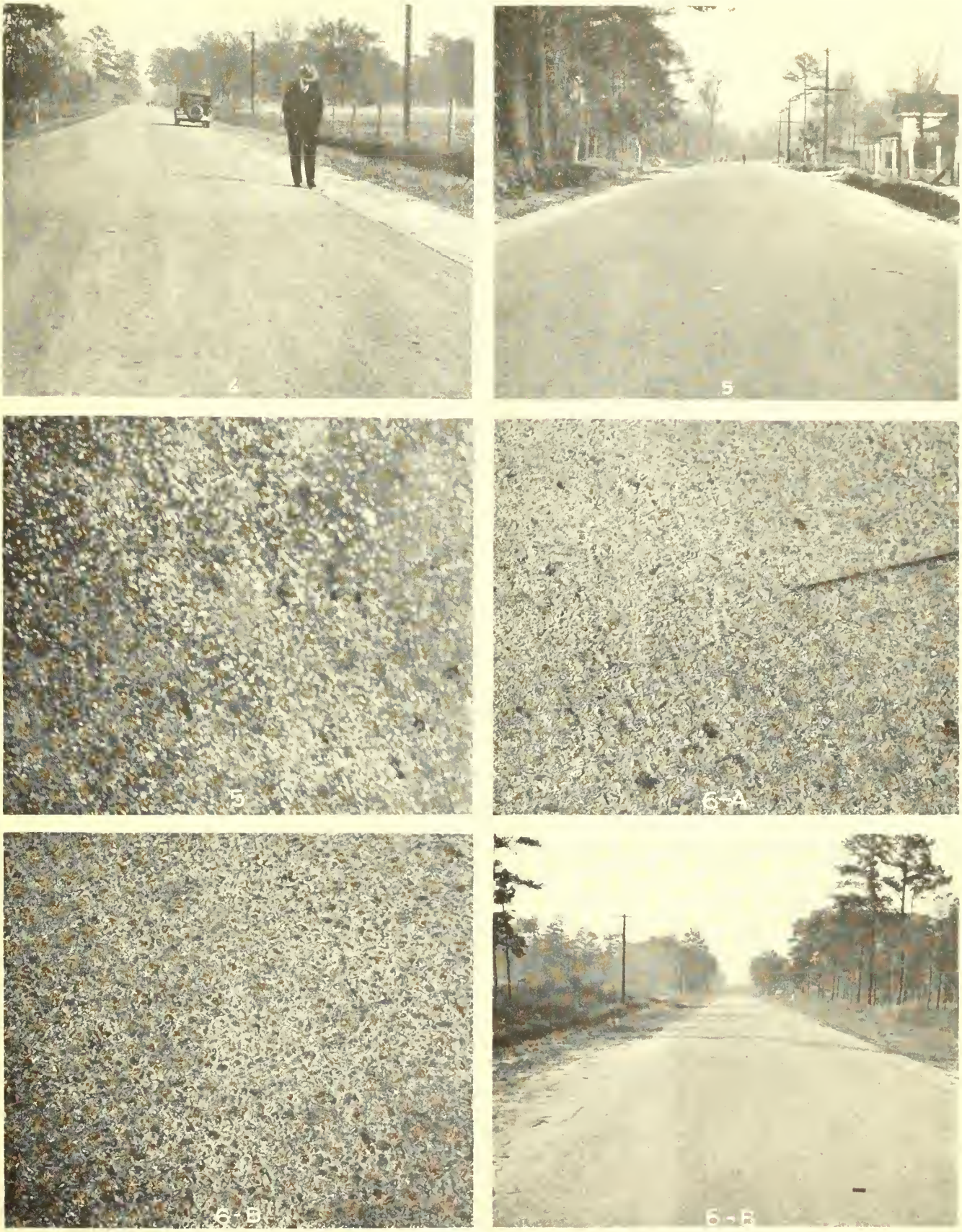


FIGURE 7.—TYPICAL EXAMPLES OF SURFACE CONDITION DURING THE SPRING OF 1931. THE NUMBERS OF THE EXPERIMENTAL SECTIONS ARE GIVEN ON THE PHOTOGRAPHS

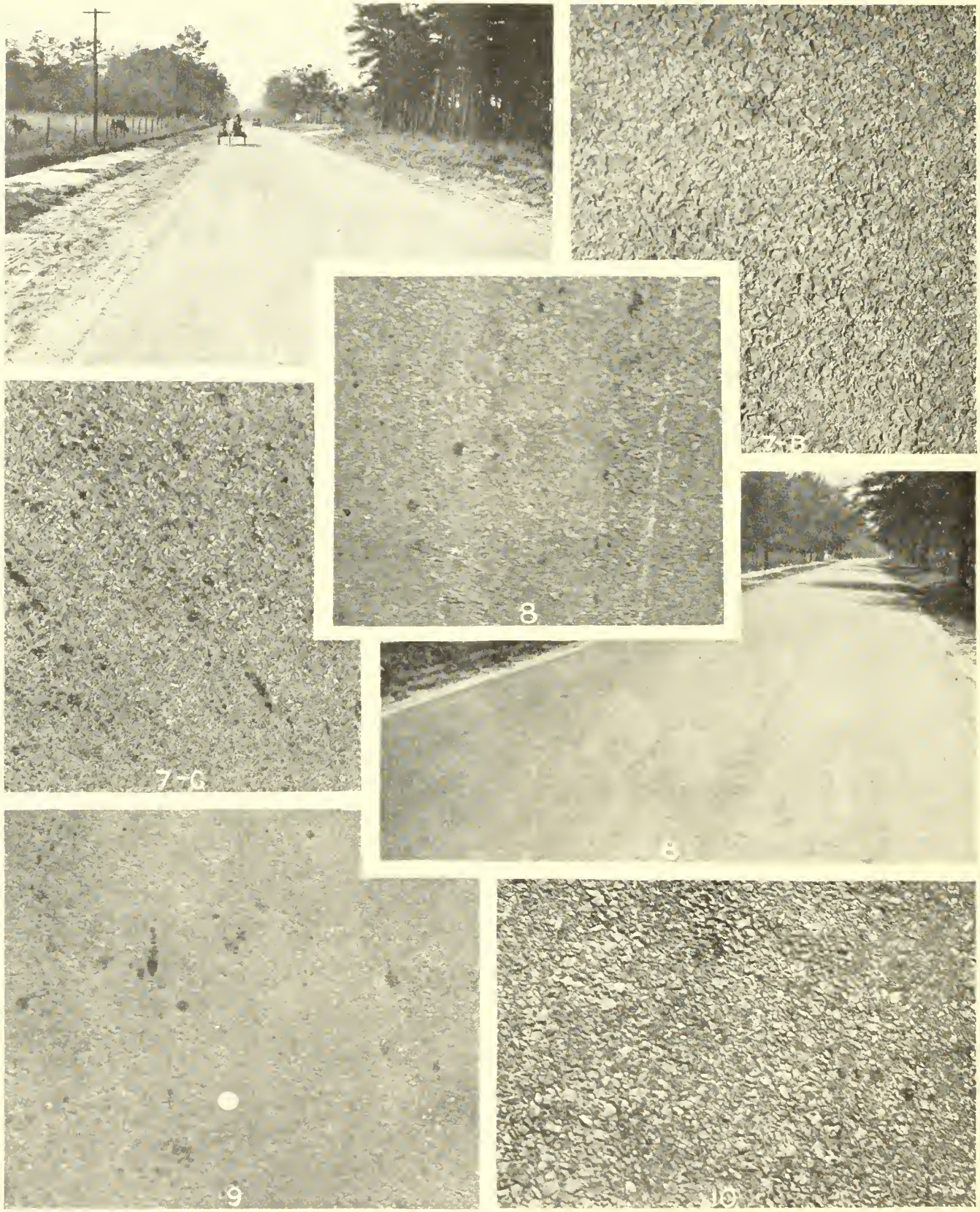


FIGURE 8.—TYPICAL EXAMPLES OF SURFACE CONDITION DURING THE SPRING OF 1931. THE NUMBERS OF THE EXPERIMENTAL SECTIONS ARE GIVEN ON THE PHOTOGRAPHS

havior of the marl base would have been more satisfactory. The application of the tar prime coat was not effective in preventing the movement of moisture through the marl base, as that portion of the base which had been penetrated by the prime was softened by moisture the same as the untreated material.

In the case of the sand-clay the condition was strikingly different, as that portion of the base which had been penetrated by the prime remained firm and hard and apparently was not affected by the action of moisture. Although the base underneath the primed surface contains a high percentage of moisture during the winter months, it has thus far retained its supporting power, so that no surface failures have occurred.

Considerable difficulty was experienced during construction because of the poorly bonded surface of this sand-clay. This material probably would not have been suitable for a sand-clay road during dry weather. However, upon application of the prime and construction of the bituminous wearing surface, enough moisture was retained in the sand-clay to bond it properly and render it entirely suitable as a base.

During recent years it has been the belief that for a clay-bonded material to serve satisfactorily as a base, the proportions of clay and granular material should be such that, as a wearing surface, it would remain stable in wet weather and have the minimum amount of raveling in dry weather. In view of the results thus far obtained from this and other similar experiments, definite indications are that the amount of clay required to prevent raveling in dry weather is excessive when used as a base for bituminous surfacing and that no more clay should be used than that amount which will produce a stable surface in wet weather.

The behavior of these experiments to date has shown conclusively that, where a surface mat is constructed on a low-type nonrigid base, a richer and more plastic mat is required than for a more rigid type. At this time there are also indications that under these same conditions a thin mat has an advantage over a thicker mat. The good behavior of the surface treatments on both types of base may not be entirely due to the thickness, as this type of surface is considerably higher in bitumen than the mixed sections. Although the percentage of bitumen used in the mixed sections would in no case equal the amount used in the surface treatment, it is believed that a higher percentage of bitumen would have resulted in more favorable behavior of these mixed sections.

Although the results thus far obtained from these experiments show little or no difference between the different grades of asphalt used, it is believed that a preference should be given to those of higher penetration where a nonrigid type base is used.

CONCLUSIONS SUMMARIZED

Although these experimental sections have not been in service long enough to warrant drawing definite conclusions, their early behavior and present surface conditions appear to justify the following statements.

1. The minimum amount of clay binder is desirable in sand-clay and gravel material when used for base construction for bituminous wearing surfaces, except under arid conditions, when a larger percentage of this material may be used. The bonding of the immediate surface of a base composed of granular material to make it suitable for the construction of the bituminous wearing surface may be accomplished by the use of a heavier prime application, together with such mixing, smoothing, and rolling as may be necessary, rather than by the addition of clay material. The maximum percentage of clay in bases for these types of surfaces should be limited to that amount which will produce a good wearing surface in wet weather.

2. Although there is considerable difference in the composition of marl, which affects its resistance to moisture, the general use of this material for base construction should be limited to areas where moisture conditions are most favorable.

3. A thin rich mat of the surface-treatment type seems to offer advantages over the thicker mat when used on a nonrigid base, because of its ability to adjust itself more readily to such movements as may occur in the base.

For a wearing surface consisting of a relatively thick bituminous mat, such as was used on the majority of these experiments, a richer, more plastic mixture is necessary than is required when the base is of a more rigid type. On this project an insufficient amount of bituminous material was used in the mixed sections.

GASOLINE TAXES—FIRST HALF OF 1931

Tax earned on motor vehicle fuel, etc., disposition of fund, and gallons taxed, from reports of State authorities

State	Gross tax assessed prior to deduction of refunds	Exemption refunds (Deducted from gross tax)	Total tax earnings on motor fuel vehicles	Other receipts under tax law (licenses)	Grand total earnings (tax and other receipts	Disposition of grand total earnings: Collection costs	Construction and maintenance of rural roads: State highways	Construction and maintenance of rural roads: Local roads	State and county road bond payments [1]	On city streets	For miscellaneous purposes	Tax rates, 1931: Cents per gallon: Jan. 1	Tax rates, 1931: Cents per gallon: June 30	Tax rates, 1931: Date of rate change	Gasoline taxed: Net gallons	Gasoline taxed: Percentage increase [2]
Alabama	$3,200,225		$3,200,225		$3,200,225	$9,118	$555,605	$1,596,732	$1,038,770			4	4		80,005,626	−6.5
Arizona	1,788,502	$163,776	1,624,726	$501,446	2,126,172	[4]	1,627,867	498,305				4	5	Jan. 30	33,194,313	−1.1
Arkansas	3,267,935	212,416	3,055,519		3,055,519	67,500		378,255	[5] 2,558,919	$50,845		5	6	Feb. 26	54,037,015	−6.3
California	20,723,606	2,212,544	18,511,062	[6] 735,000	19,246,062	38,898	12,804,776	6,402,388				3	3		617,035,384	14.0
Colorado	3,365,278	310,862	3,054,416		3,054,416	[7] 30,358	2,116,840	816,496		90,722		4	4		76,360,389	3.4
Connecticut	2,097,423	2,483	2,094,940	49,130	2,144,070	[8]	2,144,070					2	2		104,746,999	3.2
Delaware	518,976	29,611	489,365		489,365	[9]	489,365					3	3		16,312,162	4.1
Florida	7,623,982		7,623,982	[10] 14,232	7,638,214	14,232	2,541,327	423,555	[11] 2,541,327		[12] 2,117,773	6	[13] 6		127,066,367	5.4
Georgia	6,365,601		6,365,601		6,365,601	[14] 4,200	4,240,934	1,060,234			[15] 1,060,233	6	6		106,093,343	−1.0
Idaho	1,331,087	118,503	1,212,584	388	1,212,972	8,019	1,051,386		[16] 153,567			5	5		24,251,680	−1.0
Illinois	13,990,088	593,733	13,396,355		13,396,355	42,601	8,902,503	4,451,251				3	3		446,545,165	3.6
Indiana	8,997,330	532,454	8,464,876	37	8,464,913	29,047	6,326,899	1,581,725		527,242		4	4		224,413,636	12.3
Iowa	5,946,968	649,830	5,297,138		5,297,138	17,000	3,128,138	2,152,000				3	3		176,571,279	6.0
Kansas	4,854,587	1,206,815	3,647,772		3,647,772	[17]	2,747,772	900,000				3	3		121,592,400	−18.8
Kentucky	4,054,046		4,054,046		4,054,046	15,501	4,038,545					5	5		81,025,634	4.7
Louisiana	4,462,153	222	4,461,931		4,461,931	31,000	1,653,564		1,891,181		[18] 886,186	5	5		89,238,632	−.7
Maine	1,771,037	170,450	1,600,587		1,600,587	34,448	783,070	783,069				4	[19] 4		[20] 38,594,253	11.1
Maryland	3,389,018	144,346	3,244,672		3,244,672	13,573	2,584,879			[21] 646,220		4	4		81,116,787	1.8
Massachusetts	6,060,946	80,452	5,980,494		5,980,494	[22]	3,209,744	902,267		1,868,483		2	3	May 1	247,720,983	4.2
Michigan	11,065,515	1,064,407	10,001,108	18,231	10,019,339	24,558	5,492,615	2,950,000	1,500,000	35,000	[23] 17,166	3	3		[24] 333,370,258	.3
Minnesota	5,703,938	546,863	5,157,075		5,157,075	[25]	3,438,050	1,719,025				3	3		171,902,484	10.3
Mississippi	2,774,770		2,774,770	67,755	2,842,525	[26] 6,920	1,342,365	1,386,950			[27] 106,290	5	5		55,495,393	−17.7
Missouri	4,463,803	112,422	4,351,381		4,351,381	22,712	4,328,669					2	2		217,569,048	8.3
Montana	1,689,546	267,817	1,421,729		1,421,729	13,711	1,408,018					5	5		28,434,580	4.5
Nebraska	4,452,430	32,357	4,420,073		4,420,073	3,750	3,312,242	1,104,081				4	4		110,501,811	4.0
Nevada	406,645	43,191	363,454		363,454		363,454					4	4		9,086,441	21.6
New Hampshire	1,065,412	27,464	1,037,948		1,037,948		778,461		259,487			4	4		25,948,689	5.7
New Jersey	9,802,250	1,887,223	7,915,027	6,599	7,921,626	9,500	5,367,126			2,500,000	[28] 45,000	3	3		263,834,202	6.2
New Mexico	1,253,516		1,253,516	4,742	1,258,258	25,146	784,163		444,000		[29] 4,949	5	5		25,070,315	−1.8
New York	14,219,976	307,131	13,912,845	27,351	13,940,196	[30] 142,199	10,310,998	2,749,599			[31] 737,400	2	2		695,642,265	6.0
North Carolina	6,104,821	232,255	5,872,566		5,872,566	5,165	[32] 2,567,401	[32] 1,500,000	[32] 1,800,000			5	6	Apr. 1	97,876,102	−17.9
North Dakota	1,301,035	516,764	784,271		784,271	[33]	522,847	261,424				3	[34] 3		26,803,554	2.7
Ohio	18,506,660	856,906	17,649,754	400	17,650,154	[9]	9,928,212	4,412,538		3,309,404		4	4		441,243,859	−.6
Oklahoma	5,757,592	7,786	5,749,806		5,749,806	22,779	3,698,331	1,232,777			[35] 795,919	4	5	Mar. 25	134,170,623	−13.1
Oregon	3,408,036	314,435	3,093,601		3,093,601	9,409	2,395,080		689,112			4	4		77,581,843	10.8
Pennsylvania	14,281,909	83,197	14,198,712	193,367	14,392,079	117,971	10,395,090	2,379,018	1,500,000			3	3		473,131,367	18.6
Rhode Island	883,334	29,177	854,157	2,486	856,643	[36]	642,482		214,161			2	2		42,707,822	7.3
South Carolina	3,484,195	16,693	3,467,502		3,467,502	[37]	2,889,585	577,917				6	6		57,791,696	2.7
South Dakota	2,301,562	1,005,592	1,295,970		1,295,970	7,500	875,825		376,748		[38] 35,897	4	4		32,399,255	2.5
Tennessee	4,969,312		4,969,312		4,969,312	49,693	1,967,847	983,924	[39] 1,967,848			4	[40] 5		99,386,235	.6
Texas	16,004,109	862,840	15,141,269		15,141,269		11,355,952				[41] 3,785,317	4	4		378,531,742	7.7
Utah	1,070,643	21,413	1,049,230	93	1,049,323	2,309	660,064		386,950			3½	4	May 12	28,774,682	.7
Vermont	751,806		751,806		751,806		751,806					4	4		18,789,365	4.9
Virginia	5,458,080	327,315	5,130,765		5,130,765		3,591,535	1,539,230				5	5		102,581,754	4.6
Washington	5,487,693	615,033	4,872,660		4,872,660	[42]	3,120,863	1,560,431			[43] 191,366	3	5	Apr. 1	117,506,219	7.5
West Virginia	2,463,954	99,002	2,364,952	4,525	2,369,477		831,375		1,538,102			4	4		59,123,817	.9
Wisconsin	6,420,208	413,330	6,006,878		6,006,878	4,598	3,793,492	1,956,722		252,066		2	4	Apr. 1	187,172,128	6.6
Wyoming	627,866		627,866		627,866		470,900	156,966				4	4		15,696,651	12.5
District of Columbia	881,646	5,087	876,559		876,559					876,559		2	2		43,827,986	17.4
Total			244,746,853	1,625,782	246,372,635	823,415	158,332,132	48,416,879	18,860,172	10,156,541	[44] 9,783,496	Average rate 3.44 cents			7,117,874,233	4.5

[1] For State highway bonds, except as noted.
[2] Column shows percentage increase, or decrease (—), over same period last year.
[3] Increased to 5 cents on July 28, 1931.
[4] From State appropriation, $8,936.
[5] Includes $1,601,626 payments on county bonds.
[6] Refund reserve released and returned to gasoline tax fund.
[7] Includes all expenses of inspector of oils.
[8] Paid from motor vehicle license receipts, $15,000.
[9] Paid from State treasury.
[10] Includes $12,015 from dealer's licenses and $2,217 from reserve fund to cover collection cost.
[11] For county and town bonds.
[12] Includes $200,000 to permanent building fund under State board of control for buildings of higher learning, remainder to schools under county boards of public instruction.
[13] Emergency tax of 7 cents passed, and becomes effective Aug. 1, 1931, to July 1, 1933.
[14] Clerk's salary for full year taken from first collections of year.
[15] To an equalization school fund for public schools.
[16] State treasury note payment.
[17] From State general fund, $7,500.
[18] Includes $443,093 for State Board of Education, and $443,093 for Boards of Commissioners of Port of New Orleans and Lake Charles Harbor.
[19] Referendum petition postponed 5 cent tax until election in September, 1932.
[20] Excludes 5,681,678 gallons taxed 1 cent for uses other than in motor vehicles.
[21] Baltimore city only.
[22] State appropriation $10,000.
[23] For aviation fund; from taxes collected from gasoline sold for airplanes, included in "other receipts."
[24] Includes 572,205 gallons sold for airplanes, tax for which is shown in "other receipts."
[25] State appropriation, $7,146.
[26] Includes expenses of State audit of accounts, $4,220.
[27] For financing sea wall to protect road, partly paid by extra gasoline taxes in tide-water counties.
[28] For inland waterways under State Department of Commerce and Navigation, partly paid for by 2 cent gas tax for motor boats.
[29] Undistributed suspense fund.
[30] Allowance of 1 per cent on gross receipts for loss in handling, and in addition $37,505 paid for collection cost from State treasury.
[31] Includes $687,400 to New York city general funds, and $50,000 to reserve for refunds.
[32] Disposition of funds approximate.
[33] From State appropriation, $12,500.
[34] Referendum petition prevented collection of 4 cent tax which will be voted upon in March, 1932.
[35] Includes $756,241 to special emergency relief fund, and $39,678 to suspense accounts.
[36] From motor-vehicle fees.
[37] From State tax commission appropriation.
[38] Reserve for refunds.
[39] On State highway bonds, $983,924 and same amount for county bonds.
[40] Increased to 6 cents on July 1, 1931.
[41] For free-school fund, according to State constitution.
[42] From motor-vehicle fund, $5,000.
[43] Reserve for refunds.
[44] Includes $7,406,416 for schools, $2,055,190 for special purposes (as noted) and $321,890 for suspense and reserve accounts.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.
Report of the Chief of the Bureau of Public Roads, 1929.
Report of the Chief of the Bureau of Public Roads, 1930.

DEPARTMENT BULLETINS

No. *136D Highway Bonds. 20c.

*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.

*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.

*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.

*660D. Highway Cost Keeping. 10c.

1279D. Rural Highway Mileage, Income, and Expenditures 1921 and 1922.

1486D. Highway Bridge Location.

DEPARTMENT CIRCULAR

No. 331C. Standard Specifications for Corrugated Metal Pipe Culverts.

TECHNICAL BULLETIN

No. 55T. Highway Bridge Surveys.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y. Road Work on Farm Outlets Needs Skill and Right Equipment.

MISCELLANEOUS CIRCULARS

No. 62MC. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-Aid Highway Projects.

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109MC. Federal Legislation and Regulations Relating to the Improvement of Federal-Aid Roads and National-Forest Roads and Trails, Flood Relief, and Miscellaneous Matters.

MISCELLANEOUS PUBLICATION

No. 76MP. The Results of Physical Tests of Road-Building Rock.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio. (1927)

Report of a Survey of Transportation on the State Highways of Vermont. (1927)

Report of a Survey of Transportation on the State Highways of New Hampshire. (1927)

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio. (1928)

Report of a Survey of Transportation on the State Highways of Pennsylvania. (1928)

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.

Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.

Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.

Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

OCTOBER 31, 1931

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE			Estimated total cost	Federal aid allotted	MILEAGE				
				Initial	Stage	Total			Initial	Stage	Total		
Alabama	2,357.4	$ 2,752,593.91	$ 1,341,440.01	99.8	12.2	111.1						$ 5,323,972.91	Alabama
Arizona	1,043.4	4,429,904.62	3,106,399.60	219.9	72.3	289.1	$ 216,102.75	$ 151,950.86	20.0	19.9	39.8	1,685,377.32	Arizona
Arkansas	1,880.0	4,717,650.89	2,343,177.54	114.4	86.6	200.0	382,032.53	181,016.25	17.3		17.3	1,957,801.19	Arkansas
California	2,022.1	12,971,335.22	5,255,603.52	353.5	79.8	433.1	1,486,749.49	283,779.86	25.5	3.6	29.1	4,147,203.99	California
Colorado	1,436.1	5,590,391.33	2,916,061.46	249.3	82.0	331.3	844,974.75	478,291.34	29.3	4.5	32.9	2,302,759.11	Colorado
Connecticut	276.8	3,948,540.91	1,426,527.04	34.5		34.5						697,401.80	Connecticut
Delaware	353.4	521,793.00	260,142.00	12.5		12.5						569,729.08	Delaware
Florida	554.4	6,147,899.61	2,972,456.16	174.4		174.4						3,046,471.31	Florida
Georgia	2,975.2	6,995,889.51	3,192,097.75	160.1	160.0	320.1	620,396.25	230,315.45	9.2	15.7	24.9	2,971,344.42	Georgia
Idaho	1,421.9	2,190,639.09	1,310,913.68	139.7	82.7	222.4	468,906.90	232,886.59	13.8	7.4	21.0	1,475,512.52	Idaho
Illinois	2,537.5	22,839,763.74	10,446,911.19	680.1	29.7	709.8	5,135,029.04	2,332,334.60	164.3	4.7	199.0	3,544,043.42	Illinois
Indiana	1,630.9	11,116,657.29	5,464,159.40	345.6		345.6	1,729,696.47	824,485.40	62.6	12.5	75.0	2,548,183.74	Indiana
Iowa	3,314.8	2,376,684.14	950,411.47	55.4	22.3	77.7						2,842,419.77	Iowa
Kansas	3,567.0	4,283,194.25	2,010,573.24	230.5	29.9	259.3	16,609.05	9,304.02	2.9		2.9	3,065,290.49	Kansas
Kentucky	1,711.6	4,103,393.25	1,997,832.32	238.9	60.0	298.9	112,797.25	56,398.61	3.2	4.0	7.2	2,017,274.55	Kentucky
Louisiana	1,455.0	8,306,967.77	3,691,415.08	189.1	20.3	209.4	2,567,840.55	135,591.56	1.0		1.0	1,580,926.58	Louisiana
Maine	866.8	3,624,948.75	1,512,186.42	70.7		70.7	37,546.34	14,085.00	.9		.9	1,265,660.42	Maine
Maryland	762.2	806,057.95	331,912.28	21.3		21.3						911,089.46	Maryland
Massachusetts	754.8	11,923,157.53	3,579,081.86	94.6		94.6	1,796,074.38	483,330.15	17.6		17.6	1,280,102.69	Massachusetts
Michigan	1,865.6	11,117,448.45	4,868,444.98	375.1	50.3	425.4	664,679.15	282,339.07	37.0		37.0	4,385,828.30	Michigan
Minnesota	3,957.6	8,514,289.67	3,684,152.47	79.5	318.0	397.5	769,974.64	246,000.00	38.1	14.3	52.4	2,734,307.97	Minnesota
Mississippi	1,782.2	4,025,405.52	1,973,890.24	184.3	70.6	254.9	242,383.97	119,760.39		12.5	12.6	6,594,583.56	Mississippi
Missouri	2,867.3	4,961,593.49	2,049,321.97	135.5	20.9	156.4						3,644,716.24	Missouri
Montana	2,381.0	9,409,259.86	4,719,276.84	533.9	72.1	605.9	1,045,177.24	587,946.47	99.3	28.0	126.3	2,940,090.62	Montana
Nebraska	3,999.1	9,493,691.01	4,012,001.98	295.1	146.4	441.5	25,464.82	12,727.40		19.3	19.3	2,575,697.73	Nebraska
Nevada	1,155.7	2,772,153.01	1,985,134.85	99.4	212.9	311.3	4,951.97	4,399.73		.5	.5	1,553,868.48	Nevada
New Hampshire	407.4	1,101,793.55	407,090.45	21.7	4.3	26.0						529,375.00	New Hampshire
New Jersey	573.9	6,227,244.82	2,160,299.27	67.8	.6	68.2	660,151.74	275,075.87		.2	.2	1,599,922.94	New Jersey
New Mexico	2,245.9	1,625,133.91	1,065,562.06	67.4	11.7	79.1	127,291.79	80,452.98	2.4		2.4	1,845,211.88	New Mexico
New York	2,871.7	30,538,380.33	12,199,747.60	631.3		631.3	1,624,800.00	644,000.00	36.5		36.5	5,439,782.04	New York
North Carolina	2,184.0	2,321,261.35	1,141,703.19	71.6	4.0	75.6	413,800.69	196,212.82	19.2	11.5	30.7	3,954,399.91	North Carolina
North Dakota	4,949.4	2,359,013.05	1,191,109.96	237.8	264.7	502.5	780,313.54	405,639.84	102.1	138.7	240.9	2,242,271.96	North Dakota
Ohio	2,665.3	13,394,647.41	4,806,622.57	233.3	35.1	268.4	1,383,489.96	493,111.27	30.6	5.7	36.3	4,555,650.29	Ohio
Oklahoma	2,138.7	6,092,197.53	2,979,239.99	170.4	105.1	275.5	4,499.14	2,482.49				2,652,189.41	Oklahoma
Oregon	1,487.0	4,603,523.94	2,586,858.52	153.1	35.6	188.7						2,141,009.20	Oregon
Pennsylvania	2,863.3	9,807,507.76	3,972,892.56	194.9		194.9	15,183.78	7,040.14	.1		.1	5,101,432.42	Pennsylvania
Rhode Island	249.7	1,097,677.93	309,140.62	14.3		14.3						662,607.52	Rhode Island
South Carolina	1,951.8	3,226,699.25	1,417,073.56	86.7	56.9	132.6	564,458.79	150,042.23	19.7	9.0	28.7	1,489,704.30	South Carolina
South Dakota	3,940.1	3,755,472.01	2,134,989.03	232.1	210.7	442.9	158,622.23	93,635.86	2.9	12.7	15.6	1,612,467.40	South Dakota
Tennessee	1,670.7	548,910.76	274,452.78	9.3		9.3	509,641.04	231,532.60	9.6	15.1	23.7	4,043,659.60	Tennessee
Texas	7,300.1	16,462,467.15	7,737,071.96	779.9	244.1	1,023.9	2,756,672.19	1,206,709.59	141.3	61.3	192.6	7,807,980.58	Texas
Utah	1,177.2	1,055,941.05	641,980.37	57.1	29.6	86.7	234,316.57	157,222.73	19.9	.2	19.1	1,466,306.01	Utah
Vermont	313.9	976,900.91	412,307.19	29.6		29.6						540,222.52	Vermont
Virginia	1,775.0	3,903,339.22	1,807,530.75	176.9	22.0	198.9	778,512.29	389,822.19	39.5	7.4	46.9	2,091,976.71	Virginia
Washington	1,140.2	2,761,665.50	1,274,313.59	77.1	18.2	95.3	491,468.47	264,400.00	17.1		17.1	2,074,145.27	Washington
West Virginia	824.1	3,702,649.67	1,610,165.06	101.2	13.8	115.0	97,329.39	40,854.06	.6		.6	1,255,505.79	West Virginia
Wisconsin	2,517.1	7,283,713.63	2,830,850.92	228.8	73.1	301.9	46,991.91	16,000.00	2.6		2.6	2,683,746.75	Wisconsin
Wyoming	1,944.3	2,423,476.72	1,360,630.38	181.9	107.9	289.9	419,067.23	234,960.96	49.9	15.2	64.0	1,350,546.63	Wyoming
Hawaii	59.3	1,084,751.98	491,061.26	31.6		31.6	82,324.99	22,869.00	1.5		1.5	2,044,977.80	Hawaii
TOTALS	95,959.9	292,866,828.39	131,690,106.69	9,027.5	2,871.9	11,899.4	29,174,171.02	11,539,774.45	1,032.0	412.9	1,444.9	125,577,242.80	TOTALS

[1] The term stage construction refers to additional work done on projects previously improved with Federal-aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 12, NO. 10 ▽ DECEMBER, 1931

MACHINE FOR DETERMINING FLEXUAL STRENGTH OF CONCRETE SLABS

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

VOL. 12, NO. 10 DECEMBER, 1931 G. P. St. CLAIR, Editor

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THE EFFECT OF MATERIALS AND METHODS OF PLACING ON THE STRENGTH AND OTHER PROPERTIES OF CONCRETE BRIDGE FLOOR SLABS

REPORT OF A COOPERATIVE INVESTIGATION CONDUCTED BY THE PORT OF NEW YORK AUTHORITY AND THE UNITED STATES BUREAU OF PUBLIC ROADS

By L. W. TELLER, Senior Engineer of Tests, and G. W. DAVIS, Associate Engineer of Tests, United States Bureau of Public Roads

IN THE field of highway bridge engineering one of the most important developments of the times is the trend toward increased span lengths. This development has been made possible, in a large measure, by new materials, new methods of construction, and new knowledge which have been made available to the bridge engineer within the past few years. While the general tendency toward greater span lengths has been evident in the design of many types of bridges, it has been particularly noticeable in that of suspension bridges. A span of 1,800 feet is a thing of the past; one of 3,500 feet has recently been opened to traffic, and another of much greater length is already planned.

With such great span lengths the possibilities for economy through a reduction in the dead load caused by the weight of the floor system are of such importance as to merit the designer's most serious consideration. Nearly three years ago, when the design for the giant bridge over the Hudson River between Manhattan at One hundred and seventy-eighth Street and Fort Lee, N. J., was being developed, this thought led the Port of New York Authority to request the Bureau of Public Roads to cooperate in the conduct of a rather extensive series of tests, the primary object of which was the development of information which would permit a comparison to be made of the relative strengths of concrete floor slabs made of several materials and placed by each of several methods.

The tests requested were begun during the summer of 1928 and have been completed only quite recently, because of the time required to make certain extended durability tests.

SCOPE OF TESTS OUTLINED

The materials which it was desired to compare were:

1. Coarse aggregates—
   - *a*. Haydite (an artificial lightweight aggregate).
   - *b*. Rounded siliceous gravel.
   - *c*. Crushed limestone.
2. Fine aggregates—
   - *a*. Haydite.
   - *b*. Natural quartz sand.

The methods of consolidation used in the placing of the concrete were as follows:

1. Hand placing.
2. Vibrolithic process.
3. Vibration with an electric vibrator.
4. Vibration with an air hammer.

The detailed descriptions of the materials and of the methods of consolidation are given later in the report.

To provide satisfactory specimens for the tests it was necessary to construct a series of slabs sufficient in size to permit all operations to be carried out as they would be in actual construction. Since, in some cases, vibration was to be used for the consolidation of the concrete, it was necessary that the method of supporting the slabs during construction be comparable to that which would normally be used in the design of such floor systems.

The series consisted of 11 slabs, in duplicate, or a total of 22 slabs each 22 feet long, 6 feet wide, and 8 inches thick. In their construction the various materials and methods were combined as shown in Table 1. It will be seen from this table that in placing the concrete containing the Haydite aggregates the air hammer was not used. This omission was made because it was believed that a satisfactory comparison between methods could be made on the other two materials and that sufficient data concerning Haydite would result from the slabs made by the other three methods.

TABLE 1.—*List of test sections*

Slab numbers	Method of placing	Aggregates
13 and 24	Hand placed	Crushed limestone and natural sand.
12 and 23	do	Gravel and natural sand.
3 and 14	do	Fine and coarse Haydite.
5 and 16	Vibrolithic process [1]	Crushed limestone and natural sand.
4 and 15	do	Gravel and natural sand.
6 and 17	do	Fine and coarse Haydite.
8 and 19	Vibrated with electric tamper	Crushed limestone and natural sand.
7 and 18	do	Gravel and natural sand.
9 and 25	do	Fine and coarse Haydite.
11 and 22	Vibrated with air hammer	Crushed limestone and natural sand.
10 and 21	do	Gravel and natural sand.

[1] To each of the 6 slabs placed by the Vibrolithic process there was added trap rock in the proportion of 50 pounds per square yard.

The concrete in each of these slabs was subjected to the following tests:

1. Flexural strength—
   - *a*. Flexural strength of 36 by 72 inch slabs, 8 inches thick.
   - *b*. Flexural strength of 8 by 8 by 36 inch sawed beams.
2. Compressive strength—
   - *a*. Compressive strength of 6 by 8 inch drilled cores.
   - *b*. Compressive strength of 6 by 12 inch control cylinders.
3. Modulus of elasticity—
   Modulus of elasticity of 6 by 8 inch drilled cores.
4. Bond strength of embedded steel—
   - *a*. Vertically placed bars.
   - *b*. Horizontally placed bars.
5. Stress and deflection measurements in reinforced slabs, 36 by 72 inches in area.
6. Absorption tests.
7. Wear tests, to determine relative surface hardness of the concrete.
8. Tests of resistance to alternate freezing and thawing.

87154—31——1

GENERAL PROCEDURE DESCRIBED

In order that the effect of construction methods might be the same on the test slabs as on actual bridge floor slabs, the test slabs were constructed on stringers which consisted of 18-inch, 54.5-pound I beams, 22 feet long, placed 5 feet 4 inches on centers and supported at either end by short concrete pedestals.

Although each of the large slabs was constructed as a unit, it was divided in the forms into sections of a size suitable for testing by wooden dividing strips or partitions about half the depth of the forms, or 3¾ inches in width. These dividing strips, 2 inches thick with chamfered edges, were fastened in the forms in a very dry condition. On placing, the wet concrete caused these strips to swell sufficiently to crack the large slabs into smaller sections. Details of the method of dividing the large slab are shown in Figures 1 and 2.

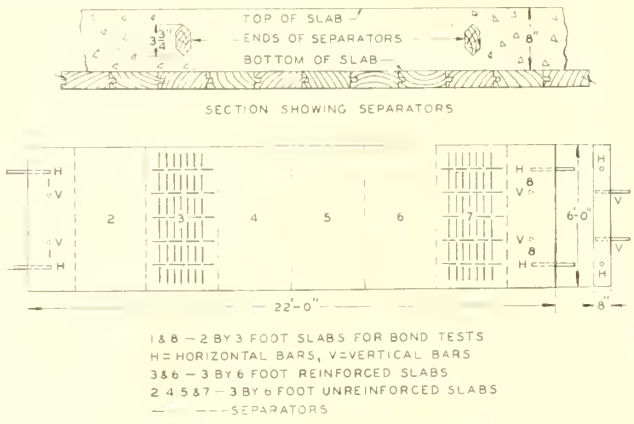

TOP OF SLAB
ENDS OF SEPARATORS
BOTTOM OF SLAB
SECTION SHOWING SEPARATORS
22'-0"
6'-0"
1 & 8 — 2 BY 3 FOOT SLABS FOR BOND TESTS
H = HORIZONTAL BARS, V = VERTICAL BARS
3 & 6 — 3 BY 6 FOOT REINFORCED SLABS
2 4 5 & 7 — 3 BY 6 FOOT UNREINFORCED SLABS
SEPARATORS

FIGURE 1.—DETAILS OF TEST SLABS

MATERIALS USED

The materials used were obtained from the same sources as the materials contemplated for use on the bridge. Their physical characteristics are given below.

Cement.—The cement used was a reputable brand of Portland cement with the following characteristics. The values given are the averages of tests of five samples.

Fineness, percentage retained on 200-mesh sieve	19. 6
Soundness	Satisfactory.
Time of set (Gilmore needle):	
Initial	3 hours, 15 minutes.
Final	7 hours, 0 minutes.
Normal consistency, per cent	21. 6
Tensile strength (pounds per square inch—1 : 3 Ottawa sand mortar):	
7 days	253 pounds.
28 days	365 pounds.

Fine aggregates.—The sand used as a fine aggregate in these tests came from Suffolk County, N. Y. It consists essentially of angular grains of quartz, containing a small amount of mica and feldspar. Physical tests gave the following results:

Sieve analysis:		
Total retained on—		
No. 4 sieve	per cent	2
No. 8 sieve	do	13
No. 14 sieve	do	24
No. 28 sieve	do	49
No. 48 sieve	do	87
No. 100 sieve	do	98
Silt and clay	do	0. 4
Apparent specific gravity		2. 67
Weight in pounds per cubic foot (dry rodded)		105
Organic matter (color test)		Satisfactory.
Strength ratio—		
7 days	per cent	116
28 days	do	106
Fineness modulus		2. 73

Haydite is a manufactured aggregate consisting of fused clay or shale. It was obtained from Jackson County, Mo. Haydite

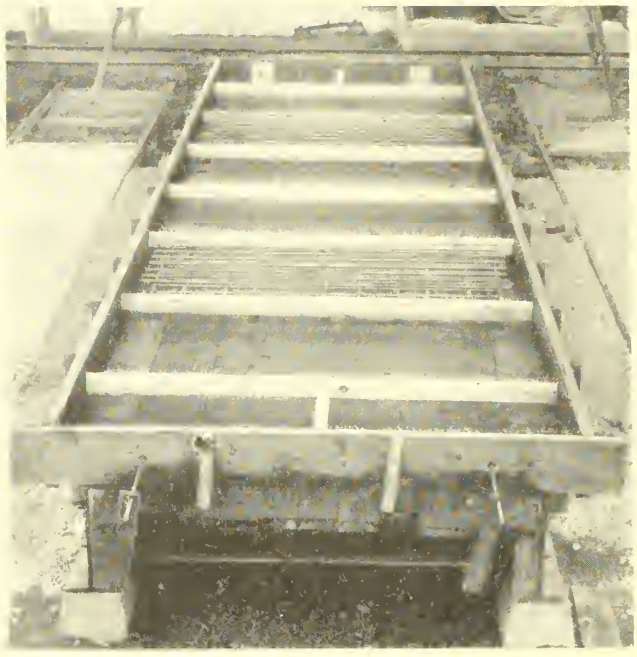


FIGURE 2.—FORM READY FOR CONSTRUCTION OF LARGE SLAB. NOTE SEPARATORS, STEEL MATS, AND BARS FOR BOND TESTS

No. 6, which was used as a fine aggregate, gave the following test results:

Sieve analysis:		
Total retained on—		
No. 4 sieve	per cent	0
No. 8 sieve	do	5
No. 16 sieve	do	37
No. 30 sieve	do	60
No. 50 sieve	do	74
No. 100 sieve	do	83
Apparent specific gravity		1. 61
Weight in pounds per cubic foot (dry rodded)		68. 75
Percentage absorption		14. 7
Fineness modulus		2. 59

Coarse aggregates.—The gravel, which was obtained from Suffolk County, N. Y., consisted essentially of rounded fragments of sandstone and quartz. Prior to the mechanical analysis the gravel as received was separated into four sizes and recombined as follows:

Over 1¼ inches	per cent	23
¾ to 1¼ inches	do	37
¼ to ¾ inch	do	32
Less than ¼ inch	do	8
Mechanical analysis		
Total retained on—		
1½-inch screen	do	9
¾-inch screen	do	52
⅜-inch screen	do	78
No. 4 sieve	do	95
No. 8 sieve	do	99
Apparent specific gravity		2. 61
Weight in pounds per cubic foot (dry rodded)		112
Percentage absorption		0. 28
Fineness modulus		7. 33

Following are the results of physical tests on the crushed stone, a siliceous limestone from Rockland County, N. Y.

Mechanical analysis:		
Total retained on—		
1½-inch screen	per cent	3
¾-inch screen	do	52
⅜-inch screen	do	88
No. 4 sieve	do	98
No. 8 sieve	do	99
No. 16 sieve	do	100
Apparent specific gravity		2. 82
Weight in pounds per cubic foot (dry rodded)		106
Percentage absorption		0. 05
Percentage wear		4. 5
Fineness modulus		7. 40

Following are the results of physical tests on the ¾-inch Haydite which was used as coarse aggregate:

Mechanical analysis:

Total retained on—		
1½-inch screen	per cent	0
¾-inch screen	do	1
⅜-inch screen	do	34
No. 4 sieve	do	85
No. 8 sieve	do	96
No. 16 sieve	do	100
Apparant specific gravity		1. 31
Weight in pounds per cubic foot (dry rodded)		51. 75
Percentage absorption		7. 4
Fineness modulus		6. 16

The trap rock which was used in the Vibrolithic concrete was obtained from Rockland County, N. Y. Following are its characteristics:

Size: 1 to 2 inches.	
Apparent specific gravity	2. 83
Percentage absorption	0. 18
Description: Gabbroitic diabase.	

DESIGN OF CONCRETE MIXES

The mixtures used were modifications of experimental mixes for the particular aggregates used, as developed in the testing laboratory of the Port of New York Authority. In both the gravel and the limestone mixes the grading of the aggregates was in accordance with specifications of the Port of New York Authority, as were the fineness moduli of the combined aggregates. For these two aggregates the real mix was identical, although the percentage of fine to coarse aggregate was varied slightly to give the desired workability. In the case of the Haydite aggregates the mix was fixed as a 1:1½:3 loose volume mix, with 3 pounds of Celite added to each sack of cement, this having been found the most satisfactory of several experimental mixes by the laboratory of the Port of New York Authority. The high absorption of Haydite made it necessary to use the material in a saturated condition in order to control the water cement ratio. Both the fine and the coarse Haydite, packed in burlap sacks, were completely submerged in water for from 16 to 20 hours. The sacks of material were then removed from the water and allowed to drain for one hour before mixing was begun.

Surface moisture determinations were made for both Haydite aggregates and for the sand just prior to their use, and an allowance for it was made in calculating the water-cement ratios. As both the gravel and the stone aggregates were sun-dried, sacked, and stored under cover, no measurable surface moisture was present. All mixtures were made as dry as compatible with the methods of placing used, the consistency of all mixes being limited to a slump of 1½ inches. Table 2 gives the complete mixing data. The batch quantities shown are the actual weights of the materials used, including surface moisture.

TABLE 2.—*Concrete design data for bridge floor slabs*

Slab No.	Surface moisture: Fine aggregate	Surface moisture: Coarse aggregate	Batch quantities: Water	Batch quantities: Cement	Batch quantities: Fine aggregate	Batch quantities: Coarse aggregate	Water-cement ratio	Real mix	Nominal mix	Cement factor	Percentage fine aggregate	Percentage coarse aggregate	Fineness modulus of mixed aggregate	Type of aggregate and method of placing
	Per cent	*Per cent*	*Pounds*	*Pounds*	*Pounds*	*Pounds*				*Bags per cubic yard*				
3	13.9	4.2	⅜–31.5 ⅞–34.5	94	100.1	150.6	0.83 .88	1:3.88	1:1.25:2.79	7.96	30.9	69.1	5.06	Haydite, hand placed.
4	1.8	0	49.4	94	225.0	420.0	.86	1:4.79	1:2.10:3.75	5.03	35.9	64.1	5.68	Gravel, vibrolithic.
5	1.9	0	51.7	94	243.0	383.0	.90	1:4.65	1:2.27:3.61	5.27	38.6	61.4	5.60	Stone, vibrolithic.
6	14.8	2.9	28.5	94	97.1	151.1	.76	1:3.88	1:1.20:2.84	7.76	29.7	70.3	5.10	Haydite, vibrolithic.
7	1.8	0	49.4	94	230.5	430.5	.86	1:4.92	1:2.16:3.84	5.26	36.0	64.0	5.67	Gravel, electric tamper.
8	1.4	0	53.2	94	258.0	402.0	.91	1:4.91	1:2.42:3.79	5.39	39.0	61.0	5.58	Stone, electric tamper.
9	14.3	4.2	½–28.5 ½–28.0	94	103.4	152.3	.80 .79	1:3.95	1:1.29:2.82	7.94	31.4	68.6	5.04	Haydite, electric tamper.
10	2.2	0	48.2	94	231.5	430.5	.85	1:4.92	1:2.16:3.84	5.18	36.0	64.0	5.67	Gravel, air hammer.
11	2.8	0	49.4	94	262.0	402.0	.91	1:4.91	1:2.43:3.79	5.28	39.1	60.9	5.57	Stone, air hammer.
12	2.5	0	47.6	94	232.6	430.5	.86	1:4.92	1:2.16:3.84	5.02	36.0	64.0	5.67	Gravel, hand placed.
13	2.9	0	49.1	94	262.8	402.8	.91	1:4.92	1:2.43:3.80	5.20	39.0	61.0	5.58	Stone, hand placed.
14	13.3	4.7	28.0	94	102.5	149.2	.78	1:3.88	1:1.29:2.75	8.18	31.9	68.1	5.02	Haydite, hand placed.
15	1.8	0	49.4	94	230.5	430.5	.86	1:4.92	1:2.16:3.84	4.98	36.0	64.0	5.67	Gravel, vibrolithic.
16	2.5	0	50.2	94	261.8	402.8	.91	1:4.92	1:2.43:3.80	5.00	39.0	61.0	5.58	Stone, vibrolithic.
17	13.9	4.5	⅜–28.0 ⅛–27.5 ½–27.0	94	104.8	150.7	.79 .78 .78	1:3.93	1:1.31:2.78	7.72	32.0	68.0	5.02	Haydite, vibrolithic.
18	1.8	0	49.4	94	230.5	430.5	.86	1:4.92	1:2.16:3.84	5.18	36.0	64.0	5.67	Gravel, electric tamper.
19	1.7	0	52.4	94	259.5	402.8	.91	1:4.92	1:2.43:3.80	5.43	39.0	61.0	5.58	Stone, electric tamper.
21	1.8	0	49.4	94	230.5	430.5	.86	1:4.92	1:2.16:3.84	5.11	36.0	64.0	5.67	Gravel, air hammer.
22	2.6	0	50.0	94	262.0	402.8	.91	1:4.92	1:2.43:3.80	5.29	39.0	61.0	5.58	Stone, air hammer.
23	2.6	0	47.2	94	232.9	430.5	.85	1:4.92	1:2.16:3.84	5.15	36.0	64.0	5.67	Gravel, hand placed.
24	2.5	0	50.2	94	261.8	402.8	.91	1:4.92	1:2.43:3.80	5.11	39.0	61.0	5.58	Stone, hand placed.
25	9.4	6.5	27.0	94	93.8	149.6	.73	1:3.78	1:1.24:2.70	8.43	31.5	68.5	5.04	Haydite, electric tamper.

SEPARATE AGGREGATES

	Fineness modulus	Weight per cubic foot	Specific gravity	Percentage voids (calculated)	Absorption
		Pounds			*Per cent*
Sand	2.73	105.00	2.67	37.0	1.00
Gravel	7.33	112.00	2.61	31.2	.28
Limestone	7.40	106.00	2.82	39.8	.05
Trap			2.83		.18
Haydite No. 6	2.59	68.75	1.61	31.6	14.70
Haydite, coarse	6.16	51.75	1.31	36.7	7.46

MIXED AGGREGATES

	Weight per cubic foot	Shrinkage factor
	Pounds	
Sand and gravel	133.9	0.82
Sand and limestone	134.1	.79
Haydite	69.6	.96

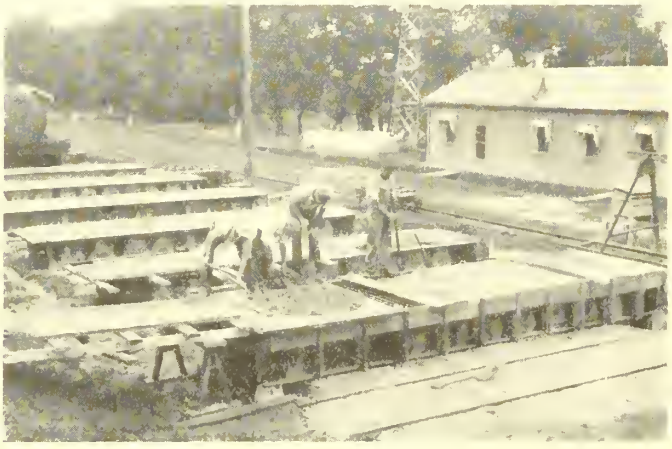
METHODS OF CONSTRUCTION

In the construction of the test slabs six forms were used. These forms, constructed of 2-inch tongue-and-groove planking, were set up with 8-foot intervals between them. Down the center of these 8-foot spaces narrow-gage tracks were laid running parallel to the long axes of the slabs. Perpendicular to these tracks and passing along one end of the forms, there was laid another narrow-gage track which led to a field testing

machine designed to make cross-bending tests on the 3 by 6 foot sections. Narrow-gage cars with platform decks fitted with rollers were used to move the test specimens.

All aggregates were stored and all mixing was done on a terrace or bench about 4 feet above the level of the forms.

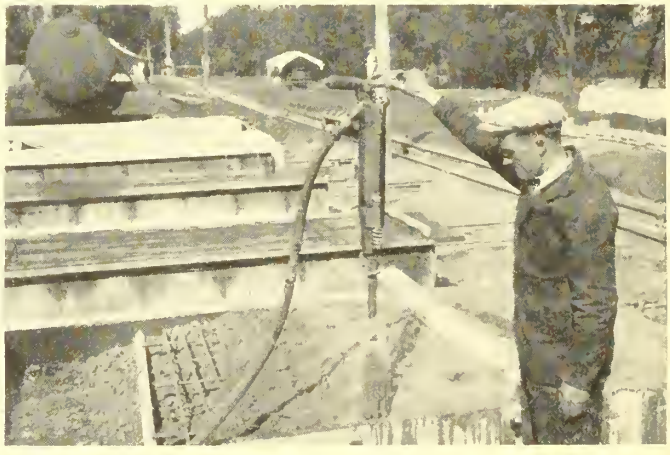
The mixing was done in a 10-cubic-foot drum-type mixer electrically driven at the rate of 20 r. p. m. All batches were mixed for two minutes and then discharged into concrete buggies. The buggies were wheeled to a platform resting on top of the form, where they were dumped. The concrete was shoveled from this platform into place in the slab forms and compacted and finished by one of the following methods.



HAND PLACING

VIBROLITHIC PROCESS

ELECTRIC TAMPER



AIR HAMMER

FIGURE 3.—THE FOUR METHODS USED FOR CONSOLIDATING THE CONCRETE

Hand placing.—In the case of the hand-placed slabs the concrete was well spaded along the edges of the forms, tamped across once with an 8 by 2 inch steel-edged strike board, and then struck off. The slab was then belted with an 8-inch rubber belt and covered with wet burlap as soon as the surface would allow.

Vibrolithic process.—The concrete in these slabs was placed and struck off in the same manner as in the case of the hand-placed slabs. As soon as the surface of the slab had been struck off it was covered uniformly with trap rock, 1 to 2 inches in size, in the proportion of 50 pounds to the square yard of surface. On this stone special slatted platforms, 6 feet long by 14 inches wide, were placed. Over these platforms the patented vibrator was rolled and the concrete subjected to about 2,200 vibrations per minute for a period of three to four minutes for each 6-foot section. As the forms were filled the platforms were moved forward in steps of approximately 3 feet and the concrete vibrated anew, five positions of the platforms covering the entire 22-foot slab. The irregular mortar surface left on removal of the platforms was smoothed off with a wooden strike board and the surface finished by belting. These slabs were also covered with wet burlap.

Two types of platform were used in these tests. The first series of Vibrolithic slabs was finished with the standard platform which has ½-inch spaces between the cleats. On the platforms used in the second series the cleat spacing was reduced to one-fourth inch in an attempt to reduce the amount of mortar brought to the surface. This modification apparently reduced somewhat the thickness of the mortar cover.

The electric tamper.—The placing of concrete by this method followed the procedure used in the case of the hand-placed slabs up to and including the shoveling of the concrete from the dumping platform into the forms. The concrete once in place was vibrated by means of a patented electric tamper which causes a vibration of relatively small displacement but of high frequency, quite different from the direct but slower blows of the air hammer. The tamper consists essentially of an unbalanced weight on the shaft of a rotor revolved by an electric motor. When this rotor is revolved at its normal speed of 3,600 r. p. m. the unbalanced weight sets up vibrations in the entire device. These vibra-

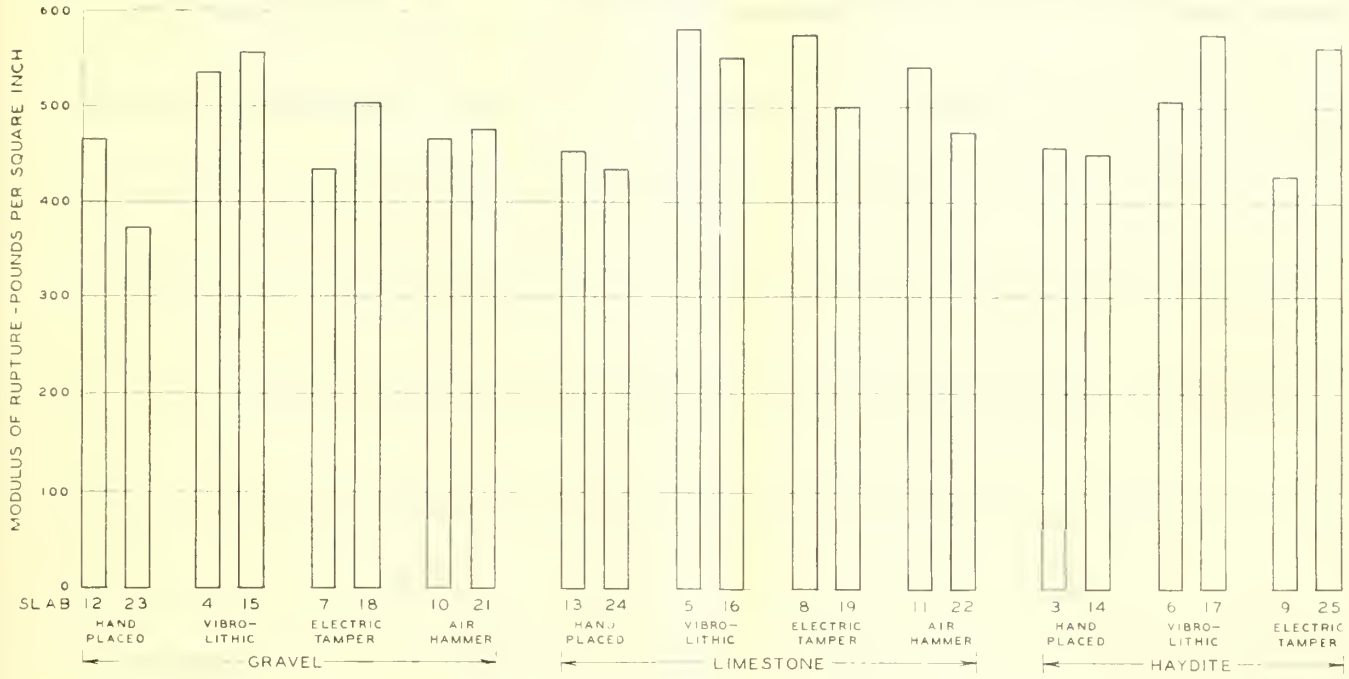


FIGURE 4.—VALUES OF MODULUS OF RUPTURE OF 36 BY 8 BY 72 INCH SLABS TESTED AT 22 DAYS. EACH VALUE IS THE AVERAGE OF FOUR TESTS

tions are transmitted to the form and to the concrete by means of a wooden tamping bar bolted to the motor frame. The vibrations are reduced before reaching the operator through spring connections between the motor and the operating handles. The total length of time that each 22-foot slab was vibrated was about 20 minutes. The slabs were finished with the wooden strike board and belted, and were covered with wet burlap immediately after the initial set.

The air hammer.—The procedure in placing this type of slab was the same as for the electric tamper, except that the concrete was compacted by vibrating the steel I-beam stringers, and through them the forms and concrete, by means of an air hammer. The hammer used was a Cleveland, type C 6 hammer, weighing 85 pounds and having a rated delivery of from 1,200 to 1,250 blows of 27 ± foot-pounds per minute when operated under 65 pounds air pressure. A wooden tamping bar 2 by 4 inches in cross section bolted to a section of a steel cutting tool was inserted in the hammer. Each I-beam was vibrated for approximately one-half a minute for each one-bag batch, the total time of vibration for the entire slab being about 20 minutes, or the equivalent of the period of vibration used in the Vibrolithic and electric tamper placements.

Figure 3 shows the equipment used in each of the four methods of placing.

Curing and storing.—At the end of 24 hours the sides and ends of the forms were removed. The slabs were kept covered with wet burlap for 12 days. On the thirteenth day after casting the decking of the form was dropped and the test specimens removed from the steel stringers. The slab sections, which had been separated by the swelling of the wooden separators in the forms, were lifted from the I beams by a specially constructed steel cradle and a chain hoist suspended from a movable steel A frame. The sections were placed in the 8-foot spaces between the forms, resting on the narrow-gage rails, which served as sills and supported the slabs clear of the ground. The form was then reassembled and was ready to be used again the following day.

FLEXURE TESTS ON 3 BY 6 FOOT SLABS

Tests for flexural strength were made on four 3 by 6 foot sections of each large slab. All of these specimens were tested at the age of 22 days, the bottom of the slab being in tension. The machine used for the testing of these specimens is described below. On the twenty-second day after casting, the four sections not containing steel reinforcement (i. e., sections 2, 4, 5, and 7 in fig. 1) were removed from the storage tracks by means of the overhead A frame and hoist and were moved on the narrow-gage railway to the field testing machine.

This field testing machine is shown on the cover. It consisted of a structural-steel frame for supporting the specimen and for providing a reaction for the ball-bearing ratchet jack used to apply the load. Freedom of motion at the knife-edges was provided. The load was applied at the third points of the span and its magnitude was determined by a measurement of the deflection of a pair of steel beams previously calibrated.

In computing the modulus of rupture of the specimen, the dead loads due to the weight of certain parts of the testing machine and to the weight of the specimen itself were considered, as were the actual dimensions of the slab at the plane of rupture.

Comparative data from these flexure tests are shown in Table 3, and the average modulus of rupture for each test slab is shown in Figure 4. These data indicate that at the age of 22 days—

1. The concrete placed by the three vibrating methods had higher flexural strength than that placed by hand methods for all of the aggregates used.

2. The concrete placed by the Vibrolithic process had higher flexural strength than that placed by any of the other methods for all of the aggregates used.

3. In the case of the gravel concrete, that placed with the air hammer and that placed with the electric

TABLE 3.—*Values of modulus of rupture of 3 by 6 foot slabs tested at age of 22 days; span, 60 inches, load applied at third points*

GRAVEL AGGREGATE

Type	Slab No.	Section No.	Modulus of rupture	Average modulus of rupture	Average weight per cubic foot
			Lbs. per sq. in.	*Lbs. per sq. in.*	*Pounds*
Hand placed	12	2	468	465	144.8
		4	474		
		5	477		
		7	442		
	23	2	306	373	145.7
		4	418		
		5	376		
		7	392		
Vibrolithic process	4	2	488	535	147.4
		4	563		
		5	569		
		7	521		
	15	2	519	554	149.9
		4	576		
		5	570		
		7	553		
Electric tamper	7	2	478	435	147.9
		4	442		
		5	440		
		7	379		
	18	2	552	504	147.6
		4	521		
		5	499		
		7	443		
Air hammer	10	2	509	466	147.9
		4	459		
		5	437		
		7	457		
	21	2	490	476	147.5
		4	469		
		5	490		
		7	454		

LIMESTONE AGGREGATE

Hand placed	13	[1] 2	441	452	148.1
		[1] 4	457		
		[1] 5	485		
		[1] 7	427		
	24	[1] 2	431	436	149.8
		[1] 4	448		
		5	451		
		[1] 7	413		
Vibrolithic process	5	2	614	579	151.8
		4	542		
		5	618		
		7	542		
	16	2	545	549	152.5
		4	525		
		5	581		
		7	546		
Electric tamper	8	2	589	574	151.2
		4	599		
		5	580		
		7	527		
	19	2	548	502	153.1
		4	515		
		5	489		
		7	457		
Air hammer	11	2	548	541	151.9
		4	586		
		5	517		
		7	512		
	22	2	538	472	150.4
		4	458		
		5	443		
		7	452		

HAYDITE AGGREGATE

Hand placed	3	[2] 2	417	456	104.1
		4	483		
		5	482		
		[2] 7	443		
	14	[2] 2	441	450	104.1
		[2] 4	441		
		[2] 5	457		
		[2] 7	459		
Vibrolithic process	6	2	463	507	105.5
		4	544		
		5	557		
		7	463		
	17	2	538	576	109.3
		4	550		
		5	557		
		7	659		
Electric tamper	9	2	425	426	105.4
		4	419		
		5	419		
		7	441		
	25	2	537	562	108.8
		4	574		
		5	592		
		7	547		

[1] Slight honeycomb in bottom. [2] Slight honeycomb.

tamper had approximately equal flexural strengths. In the case of the concrete in which crushed limestone was used as the coarse aggregate, that placed with the electric tamper had a flexural strength intermediate between that placed by the Vibrolithic process and that placed with the air hammer.

These indications are of course limited to the flexural tests at 22 days and do not take into consideration

TABLE 4.—*Values of modulus of rupture of 8 by 8 by 36 inch sawed beams; span, 30 inches, load applied at third points*

GRAVEL AGGREGATE

Type	Slab No.	Section No.	Modulus of rupture in pounds per square inch: 28 days	Average for slab	90 days	Average for slab	180 days	Average for slab	365 days	Average for slab
Hand placed	12	2	------	------	------	701	627	610	553	580
		7	------		701		593		606	
	23	2	------	427	576	554	434	476	555	547
		7	427		532		519		539	
Vibrolithic process	4	2	------	609	849	732	620	636	676	716
		7	609		615		653		755	
	15	2	------	------	------	661	750	687	718	738
		7	------		661		625		759	
Electric tamper	7	2	------	------	768	627	589	546	615	532
		7	------		486		503		449	
	18	2	------	------	------	547	550	548	623	628
		7	------		547		546		633	
Air hammer	10	2	------	475	------	627	623	545	774	590
		7	475		627		467		407	
	21	2	------	511	742	644	642	614	776	736
		7	511		547		586		696	

LIMESTONE AGGREGATE

Hand placed	13	2	------	------	681	669	521	423	480	523
		4	------		620		360		700	
		5	------		606		360		423	
		7	------		764		452		490	
	24	2	518	461	529	606	480	506	666	691
		4	------		639		515		652	
		5	------		644		503		626	
		7	404		610		525		820	
Vibrolithic process	5	2	673	616	797	773	773	710	696	710
		4	------		801		736		737	
		5	------		791		768		714	
		7	560		701		564		691	
	16	2	623	560	596	778	561	660	536	650
		4	------		885		657		643	
		5	------		876		735		739	
		7	498		754		688		684	
Electric tamper	8	2	704	630	783	752	692	647	788	745
		4	------		818		735		830	
		5	------		734		638		686	
		7	555		672		523		676	
	19	2	517	518	716	669	736	645	761	684
		4	------		661		617		710	
		5	------		700		652		596	
		7	518		598		574		667	
Air hammer	11	2	596	495	833	731	763	680	806	720
		4	------		666		652		787	
		5	------		752		634		635	
		7	393		673		672		650	
	22	2	525	501	625	622	731	619	730	678
		4	------		700		499		827	
		5	------		623		617		580	
		7	477		538		628		572	

HAYDITE AGGREGATE

Hand placed	3	2	396	401	519	469	571	496	284	305
		4	432		517		481		365	
		5	358		406		506		224	
		7	419		435		424		348	
	14	2	531	496	683	718	366	372	243	264
		4	487		758		385		259	
		5	468		788		358		280	
		7	500		643		380		274	
Vibrolithic process	6	2	526	457	580	604	491	476	245	232
		4	461		522		503		252	
		5	441		638		505		244	
		7	400		687		406		186	
	17	2	646	653	550	557	570	555	285	275
		4	628		562		553		270	
		5	678		542		-----		227	
		7	659		575		541		318	
Electric tamper	9	2	643	573	546	541	411	433	245	250
		4	549		531		485		207	
		5	576		542		464		301	
		7	524		545		372		248	
	25	2	499	503	631	553	378	407	394	355
		4	488		520		375		295	
		5	514		515		394		370	
		7	511		546		482		362	

possible effects of variations in temperature and humidity during this period.

FLEXURE TESTS ON 8 BY 8 BY 36 INCH SAWED BEAMS

The specimens used in these tests were obtained by sawing into beams the fragments of the 3 by 6 foot slabs used in the 22-day flexure tests. This operation was performed with a high-speed inserted-tooth carborundum saw which in general was very satisfactory, except that the cutting in the siliceous aggregates was somewhat slow.

applied loads and was based on the actual ruptured section of the specimen.

The flexural strengths of the individual specimens are given in Table 4, as well as averages for the beams from each section of each slab. Figure 6 shows graphically the effects of age, method of placing, and of aggregates on the flexural strength of these specimens.

In order to study the effect of the various methods of placing on the flexural strength of concrete of each material, a summary of the data in Table 4 was pre-

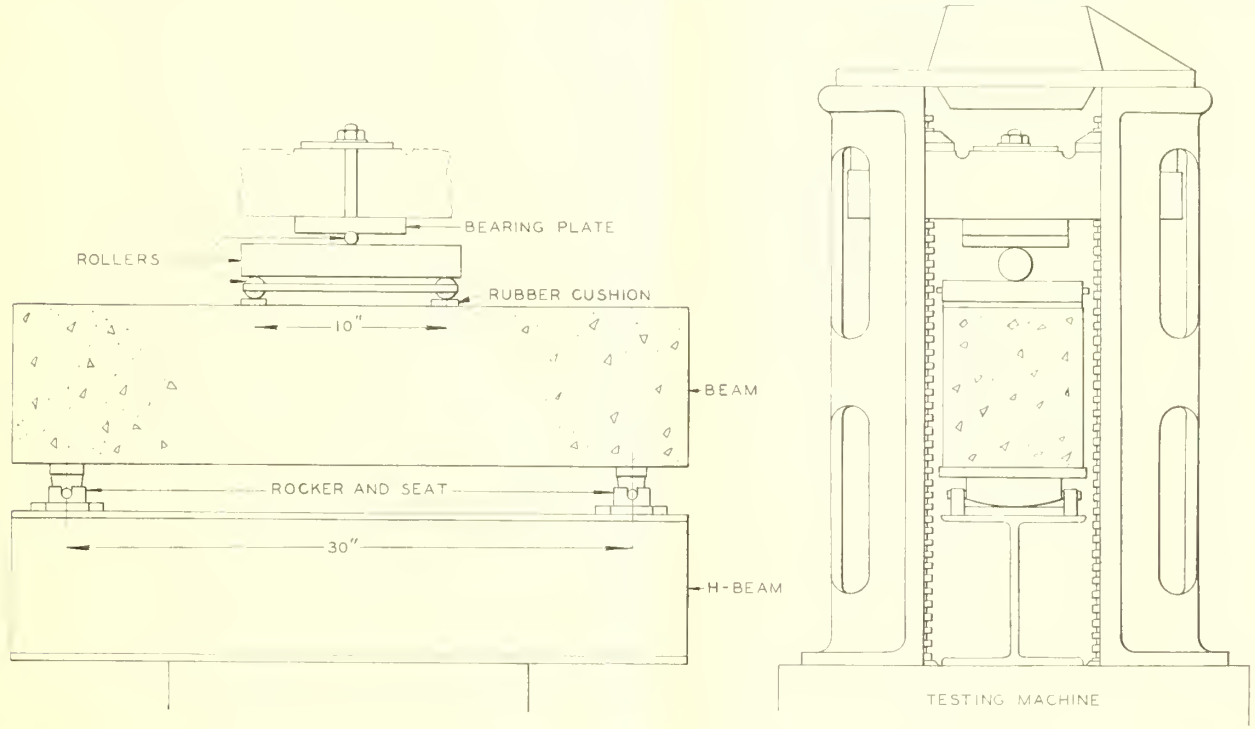


FIGURE 5.—APPARATUS USED IN MAKING FLEXURE TESTS ON SAWED BEAMS

The program called for the testing of four beams from each large slab at each of four test periods (28 and 90 days, 6 months, and 1 year). Because of the difficulty of sawing the specimens of gravel concrete and other unavoidable delays, it was not possible to prepare and test this number of specimens, and it will be noted in the tabulated data that the number of specimens varies, particularly in the 28-day tests.

Another difficulty which was not foreseen was a considerable variation in the moisture condition of the sawed specimens tested at the early ages. The sawing process requires a free flow of water over the specimen, and at the early ages some of the specimens were taken almost directly from the saw to the testing machine. Moreover, the specimens for the tests at 28 days, 90 days, and 6 months were stored in the open air without protection from the time of sawing until the time of test. In the case of the 1-year test specimens, the sawing was completed well in advance of the date of test and the specimens were stored in a metal building for at least 10 weeks before testing.

The apparatus used for this test is shown in Figure 5. The load was applied at the third points of the span, which was 30 inches. Tension was produced in the bottom of the specimen as cast. The conventional testing machine in which the apparatus was set up was operated slowly with a handwheel. The modulus of rupture was calculated from the combined dead and

pared, considering the strength of the hand-placed concrete as 100 per cent in each case. The results of this summary are shown in Table 5.

The indications of these figures are in rather striking agreement with those of the tests of the 3 by 6 foot slabs at 22 days. It will be noted that the vibrated concrete generally shows higher flexural strength than does the hand-placed concrete. This is most noticeable in the case of the specimens having the limestone aggregate, and is also clearly indicated by the specimens having gravel aggregate. In the case of the specimens in which Haydite was used, the majority of tests indicate a slight superiority on the part of the hand-placed concrete. It is also shown that of the three vibrating methods used, the Vibrolithic process yielded concrete of the greatest flexural strength.

Comment has been made on the possible effect of the moisture condition of the specimens at the time of test on their flexural strength. It is believed that the apparent general retrogression in strength between 90 and 180 days is due to the fact that the specimens were in a drier condition at 180 than at 90 days. Special care was taken with the 1-year specimens, as has been noted.

In general, the data show a normal increase in flexural strength except in the case of the Haydite concrete specimens, where a sharp drop in flexural strength took place between the 6-month and 1-year tests. The cause of this has not been determined.

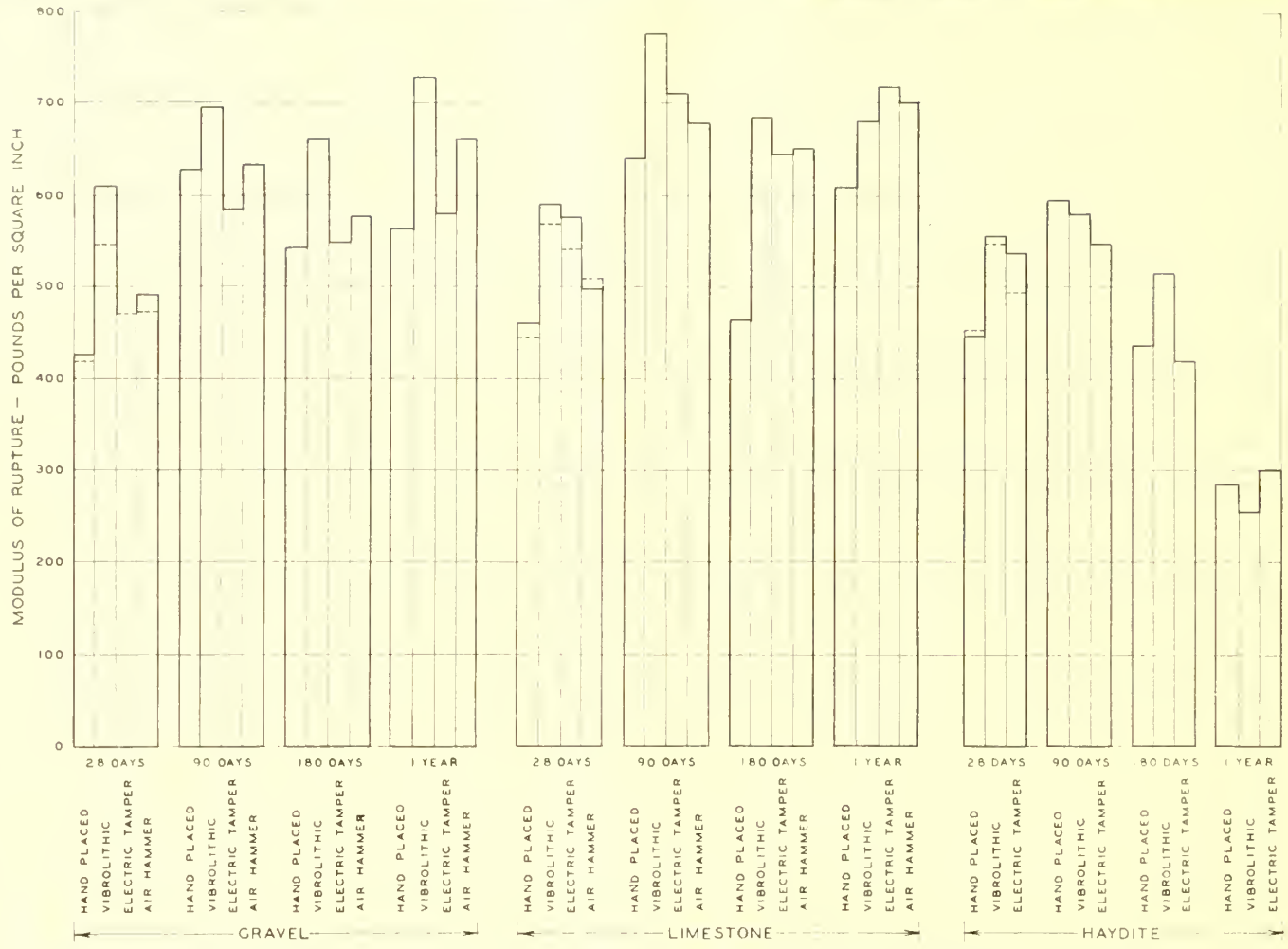


FIGURE 6.—EFFECTS OF AGE, METHOD OF PLACING, AND TYPE OF AGGREGATE ON FLEXURAL STRENGTH OF CONCRETE. VALUES SHOWN ARE MEAN VALUES OF MODULUS OF RUPTURE FOR SAWED BEAMS FROM TWO SLABS OF EACH TYPE AS GIVEN IN TABLE 4. DASH LINES INDICATE MEAN VALUES FOR LARGE SLABS BROKEN AT 22 DAYS

TABLE 5.—*Flexural strength of 8 by 8 by 36 inch sawed beams, expressed as a percentage of the strength of the beams taken from hand-placed concrete*

GRAVEL AGGREGATE

Age	Hand placed	Vibrolithic process	Electric tamper	Air hammer
DAYS	*Per cent*	*Per cent*	*Per cent*	*Per cent*
28	100	143		115
90	100	111	94	101
180	100	122	100	107
365	100	129	103	118
Average	100	126	99	110

LIMESTONE AGGREGATE

28	100	128	125	108
90	100	122	111	106
180	100	147	139	140
365	100	112	118	115
Average	100	128	123	117

HAYDITE AGGREGATE

28	100	124	120
90	100	98	92
180	100	119	97
365	100	89	106
Average	100	107	104

After the 1-year tests, additional specimens were sawed from the slabs of Haydite concrete, dried for a period of 25 days in a boiler room and tested at the age of 15 months. At this time the weight appeared to be constant. The 15-month tests showed a decrease in flexural strength over that of the specimens tested at one year. This decrease averaged about 20 per cent of the 1-year strength. Unfortunately there was not sufficient concrete available to attempt to determine the cause of this peculiar retrogression in flexural strength, but it is suspected that it may be related to the moisture-retaining properties of this material, and further investigation seems advisable.

COMPRESSIVE STRENGTH OF 6 BY 8 INCH DRILLED CORES

Tests for compressive strength were made on 6 by 8 inch drilled cores at ages of 28 days, 90 days, and 1 year. These cores were drilled with a conventional steel shot core drill from fragments of the 3 by 6 foot slabs used for the 22-day flexure tests. One core was drilled from each of the four sections of each 6 by 22 foot slab for each of the three ages at which compression tests were made. Within 24 hours after drilling the cores were capped with neat cement and stored in the open air, adjacent to the slabs from which they were taken, until they were tested. Correction was made for the length of the core in calculating the unit compressive strength.

The data from these tests are given in Table 6 and the average values are shown graphically in Figure 7.

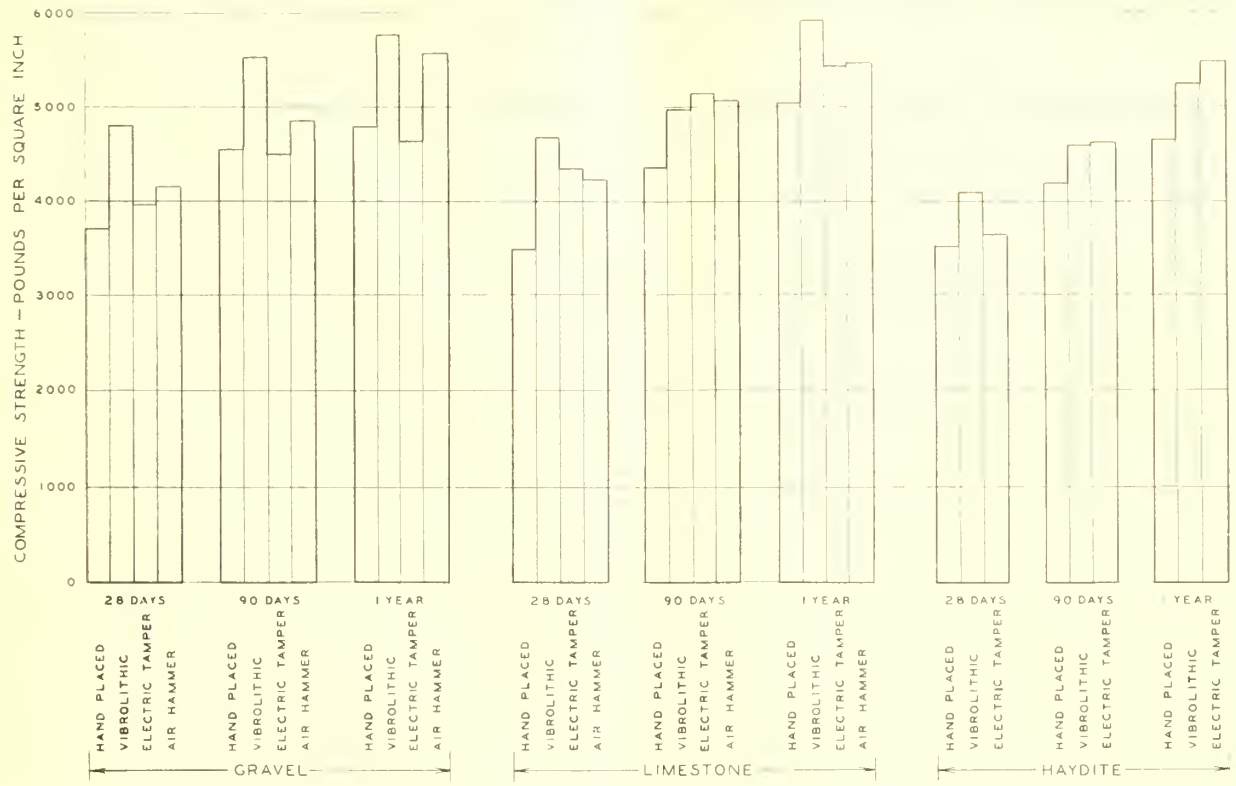


FIGURE 7.—EFFECTS OF AGE, METHOD OF PLACING, AND TYPE OF AGGREGATE ON COMPRESSIVE STRENGTH OF DRILLED CORES

TABLE 6.—*Compressive strength of 6 by 8 inch drilled cores*

Slab No.	Aggregate	Method of placing	Compressive strength in pounds per square inch		
			28 days	90 days	365 days
12	Gravel	Hand placed	4,137	4,830	4,885
23			3,310	4,302	[1] 4,740
	Average		3,724	4,566	4,812
4	Gravel	Vibrolithic process	4,760	5,366	5,915
15			4,893	5,675	5,640
	Average		4,826	5,520	5,778
7	Gravel	Electric tamper	3,975	4,410	4,525
18			3,961	4,583	4,810
	Average		3,968	4,496	4,668
10	Gravel	Air hammer	4,137	4,920	5,600
21			4,173	4,780	5,580
	Average		4,155	4,850	5,590
13	Limestone	Hand-placed	3,471	4,614	5,150
24			3,532	4,120	4,900
	Average		3,502	4,367	5,025
5	Limestone	Vibrolithic process	4,701	4,684	5,995
16			4,652	[1] 5,278	5,825
	Average		4,676	4,981	5,910
8	Limestone	Electric tamper	4,444	5,429	5,915
19			4,210	4,844	[1] 4,955
	Average		4,327	5,136	5,435
11	Limestone	Air hammer	4,527	5,277	5,440
22			3,894	4,810	[1] 5,485
	Average		4,210	5,044	5,462
3	Haydite	Hand-placed	3,863	4,375	4,880
14			3,136	4,015	4,485
	Average		3,500	4,195	4,682

[1] Average of 3 tests. All other values given are averages of 4 tests.

TABLE 6.—*Compressive strength of 6 by 8 inch drilled cores*—Con.

Slab No.	Aggregate	Method of placing	Compressive strength in pounds per square inch		
			28 days	90 days	365 day
6	Haydite	Vibrolithic process	4,014	4,590	5,365
17			4,164	4,684	5,110
	Average		4,090	4,637	5,238
9	Haydite	Electric tamper	3,555	4,841	5,558
25			4,091	4,428	5,410
	Average		3,623	4,640	5,484

It will be noted from these data that the concrete, for all methods of placing and for all three aggregates, shows a progressive increase in strength with age. It is also evident that the mechanical methods of placing, in general, result in higher compressive strength than is obtained by hand placing and also that of the three mechanical methods used the Vibrolithic process gave concrete of the highest compressive strength.

In an effort to rate the four methods of placing used in this investigation, an average flexural strength and an average compressive strength was obtained for each method. In these average values are included all ages of test and all aggregates. The average strength is expressed, in each case, as a percentage of the strength of the hand-placed concrete of comparable age and aggregate. The result of this comparison is as follows:

	Hand placed	Vibrolithic process	Electric tamper	Air hammer
Flexural strength[1] (sawed beams)	100	120	108	113
Compressive strength[1] (drilled cores)	100	119	109	113

[1] Strength expressed as a percentage.

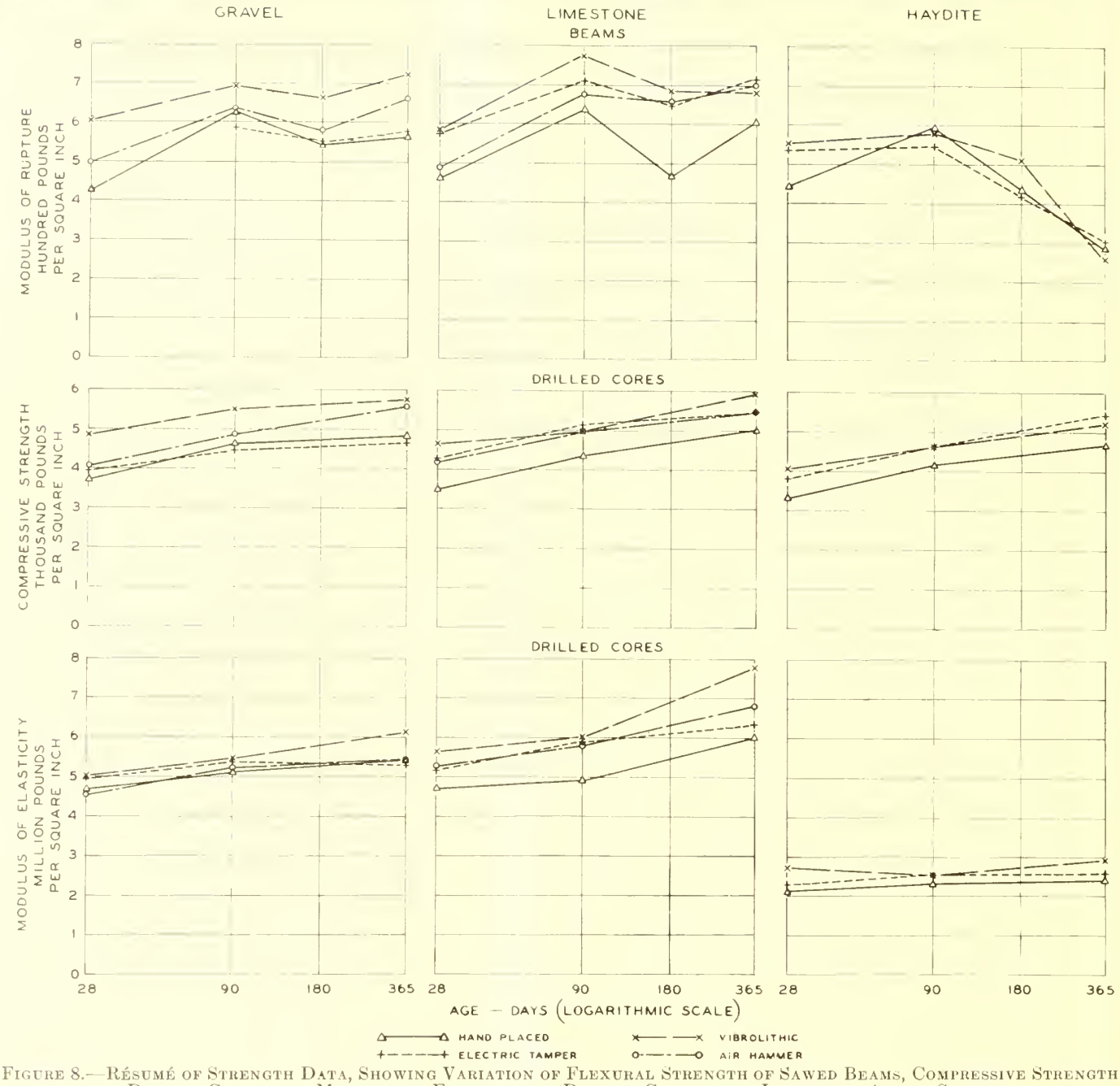


FIGURE 8.—RÉSUMÉ OF STRENGTH DATA, SHOWING VARIATION OF FLEXURAL STRENGTH OF SAWED BEAMS, COMPRESSIVE STRENGTH OF DRILLED CORES, AND MODULUS OF ELASTICITY OF DRILLED CORES, WITH INCREASING AGE OF SPECIMEN

These values show rather strikingly that the method of placing influences the flexural and the compressive strength to an equal degree. All of the data are in general accord as to the effect of the methods of placing on the strength of the concrete.

The concrete containing the Haydite aggregate compares very favorably in compressive strength with that containing the other aggregates and no retrogression in compressive strength was found to occur. A limited number of cores were tested at the age of 15 months. One half of these were in a saturated condition and the other half were thoroughly dried. No marked difference in strength between the wet and the dry specimens was found. Moreover, no appreciable change in compressive strength between the 1-year and 15-month periods was observed.

In comparing aggregates it should be remembered that the cement factor of the concrete containing the Haydite was about one and one-half times that of the concrete containing the other aggregates. If the comparison is made on the basis of equal cement factors (see Table 2) and all ages and methods of placing are averaged, it will be found that, from the standpoint of compressive strength only, the following relation exists:

Type of aggregate:	Compressive strength in pounds per square inch per bag of cement in a cubic yard of concrete
Gravel	929
Limestone	923
Haydite	560

If the reciprocals of the values of compressive strength tabulated above are obtained and multiplied by 1,000, an index is obtained which gives the number of sacks per cubic yard required per 1,000 pounds unit compressive strength in the concrete. Values of the index thus obtained are as follows:

Type of aggregate:	Bags per cubic yard for each 1,000 pounds per square inch compressive strength
Gravel	1.076
Limestone	1.083
Haydite	1.786

These values are, of course, useful only for comparing the costs of concrete of a given compressive strength when made of the materials used in this investigation.

It should also be remembered that the weight of the concrete containing the Haydite aggregate is about 105 to 110 pounds per cubic foot, as compared with a weight per cubic foot of from 145 to 150 pounds for the concrete containing the other aggregates (see Table 3), and it will probably be found in the majority of cases that decreases in slab weight are of more economic significance than are increases in the cost of construction of the slab itself.

MODULUS OF ELASTICITY

At each of the periods at which compressive strength tests were made the modulus of elasticity was determined for each of the cores. The loads were applied in a universal testing machine operated by a handwheel. The deformations were measured with a Martens type of extensometer operating on a 4-inch gage length.

TABLE 7.—*Average values of modulus of elasticity of 6 by 8 inch drilled cores*

Slab No.	Aggregate	Method of placing	Initial modulus of elasticity		
			28 days	90 days	365 days
			Lbs. per sq. in.	*Lbs. per sq. in.*	*Lbs. per sq. in.*
12	Gravel	Hand placed	4,925,000	6,105,000	5,625,000
23			4,377,000	4,165,000	[1] 5,147,000
	Average		4,651,000	5,135,000	5,386,000
4	Gravel	Vibrolithic process	5,110,000	5,876,000	6,619,000
15			4,937,000	5,042,000	5,677,500
	Average		5,023,500	5,459,000	6,148,250
7	Gravel	Electric tamper	4,280,000	5,742,000	5,549,750
18			5,680,000	5,102,000	5,100,000
	Average		4,980,000	5,422,000	5,324,875
10	Gravel	Air hammer	4,580,000	5,110,000	4,759,250
21			4,491,000	5,225,000	6,030,000
	Average		4,535,500	5,167,500	5,394,625
13	Limestone	Hand placed	4,900,000	5,135,000	6,410,000
24			4,490,000	4,617,000	5,620,000
	Average		4,695,000	4,876,000	6,015,000
5	Limestone	Vibrolithic process	5,085,000	6,007,000	7,889,750
16			6,205,000	5,970,000	7,672,500
	Average		5,645,000	5,988,500	7,781,125
8	Limestone	Electric tamper	5,305,000	6,380,000	6,750,000
19			5,010,000	5,350,000	[1] 5,893,000
	Average		5,157,500	5,865,000	6,321,500
11	Limestone	Air hammer	5,700,000	6,190,000	7,130,000
22			4,815,000	5,415,000	[1] 6,426,000
	Average		5,257,500	5,802,500	6,778,000
3	Haydite	Hand placed	2,240,000	2,401,000	2,450,500
14			2,025,000	2,290,000	2,437,000
	Average		2,132,500	2,345,500	2,443,750
6	Haydite	Vibrolithic process	2,655,000	2,642,000	3,038,000
17			2,855,000	2,394,000	2,935,000
	Average		2,755,000	2,518,000	2,986,500
9	Haydite	Electric tamper	2,230,000	2,385,000	2,600,000
25			2,352,000	2,757,000	2,660,000
	Average		2,291,000	2,571,000	2,630,000

[1] Average of 3 tests. All other values given are averages of 4 tests.

(For a description of an instrument of this type see PUBLIC ROADS, vol. 9, No. 8, October, 1928, The Modulus of Elasticity of Cores from Concrete Roads, by A. N. Johnson.) The precision of the deformation measurements is very high with this apparatus, and for the concrete used in these tests the stress-strain relation was linear up to unit stresses of several hundreds of pounds per square inch. It is the slope of this initial curve that is referred to in this report as the modulus of elasticity.

The average values for each group of four cores obtained for each age of test from each of the large 6 by 22 foot slabs are shown in Table 7, and these averages are shown graphically in Figure 8, where the effect of age on the modulus of elasticity is indicated.

This graph also contains the data from the flexure tests on the sawed beams and from the compression tests on drilled cores plotted on a logarithmic time scale and gives a good summary picture of the data which have been thus far discussed.

It will be noted in this figure that generally the value of modulus of elasticity increases with age. This effect is most marked in the case of the concrete containing the limestone aggregate and is barely noticeable in that containing the Haydite.

The modulus of elasticity of the concrete containing the Haydite aggregate is generally of about one-half the magnitude of that for the concrete containing the other aggregates. This was consistently true throughout the considerable number of tests made (72 Haydite cores). Although the slope of the load-deformation curve was radically different for this concrete, the relation appeared to be linear for moderate unit stresses except in the case of the 28-day tests, where curvature began at very low loads.[1]

The individual curves have been omitted because of their large number.

COMPRESSIVE STRENGTH OF 6 BY 12 INCH CONTROL CYLINDERS

At the time that this investigation was planned it was realized that, during the construction and curing of the several slabs, there would be weather variations and that these might have an effect on the strength of the concrete. Any appreciable effect on strength would have to be considered in drawing conclusions about the variables under study. It was believed that if a group of 6 by 12 inch cylinders were made out of the concrete that went into each slab and that if these were exposed to the same conditions of curing and weather as the slabs themselves, then for each type of aggregate, variations in strength between groups of cylinders would reflect the influence of the variations in temperature and humidity to which the concrete had been exposed.

To provide these data, five 6 by 12 inch cylinders were made from concrete from sections 2, 4, 5, 6, and 8, respectively, of each large slab. These specimens were fabricated in accordance with standard practice in steel molds resting on and covered with glass plates. When 24 hours old they were removed from the molds, capped, and placed under the wet burlap which covered the slab from which they were made. The cylinders remained on the slabs until the burlap was removed and were then stored in the vicinity until they were ready for test at the age of 28 days.

All of the data pertaining to the cylinders are given in Table 8. The water-cement ratio shown is a calcu-

[1] For a discussion of the effect of this low modulus of elasticity, see Jour. Amer. Conc. Inst., vol. 2, No. 2, October, 1930, p. 151, Construction and Design Features of Haydite Concrete, by F. E. Richart and V. P. Jensen.

lated value and the air temperature is a mean value for the 10-day period immediately following the placing of the concrete. Mean compressive strengths are shown for each group of five specimens, and the individual variations from the group means are tabulated.

TABLE 8.—*Test data pertaining to the 6 by 12 inch control cylinders*

GRAVEL AGGREGATE

Slab No.	Section No.	Water-cement ratio	10-day temperature	Compressive strength: Individual	Compressive strength: Average	Variation from group average
			° F.	Lbs. per sq. in.	Lbs. per sq. in.	Per cent
	2			3,450		8.6
	4			3,860		2.2
4	5	0.86	69.2	3,900	3,776	3.3
	6			3,640		3.6
	8			4,030		6.7
	2			3,470		5.0
	4			3,110		5.9
7	5	.86	53.2	3,350	3,306	1.3
	6			3,240		2.0
	8			3,360		1.6
	2			4,135		7.4
	1			3,905		1.4
10	5	.85	58.3	3,895	3,850	1.1
	6			3,760		2.3
	8			3,555		7.7
	2			3,766		.2
	4			3,788		2.4
12	5	.86	62.7	3,843	3,698	3.9
	6			3,745		1.1
	8			3,406		7.9
	2			3,851		2.5
	4			4,256		13.3
15	5	.86	64.7	3,662	3,757	2.3
	6			3,378		10.1
	8			3,638		3.2
	2			3,246		.4
	4			3,229		.2
18	5	.86	60.9	3,248	3,234	.4
	6			3,246		.4
	8			3,199		1.1
	2			3,791		5.2
	4			3,451		4.3
21	5	.86	43.5	3,617	3,605	.3
	6			3,681		2.1
	8			3,483		3.4
	2			2,964		2.5
	4			2,942		3.2
23	5	.85	51.6	3,258	3,040	7.2
	6			2,990		1.6
	8			3,045		.2

LIMESTONE AGGREGATE

	2			3,200		3.0
	4			3,300		0
5	5	0.90	64.8	2,940	3,300	10.9
	6			3,660		10.9
	8			3,400		3.0
	2			3,645		.7
	4			3,610		1.7
8	5	.91	52.8	4,025	3,672	9.6
	6			3,490		5.0
	8			3,590		2.2
	2			3,345		6.3
	4			3,700		3.6
11	5	.91	60.9	3,675	3,570	2.9
	6			3,580		.3
	8			3,550		.6
	2			4,380		13.3
	4			3,440		11.0
13	5	.91	62.9	4,065	3,865	5.2
	6			4,030		4.3
	8			3,410		11.8
	2			3,499		11.0
	4			3,175		.7
16	5	.91	64.1	3,056	3,153	3.1
	6			2,263		28.2
	8			3,773		19.7
	2			3,316		7.3
	4			2,742		11.3
19	5	.91	54.8	3,087	3,090	.1
	6			3,412		10.4
	8			2,893		6.4
	2			3,023		2.1
	4			3,190		7.7
22	5	.91	47.4	2,891	2,961	2.4
	6			2,783		6.0
	8			2,920		1.4
	2			3,520		15.4
	4			2,980		2.0
24	5	.91	47.4	3,117	3,051	2.2
	6			2,888		5.3
	8			2,752		9.8

TABLE 8.—*Test data pertaining to the 6 by 12 inch control cylinders*—Continued

HAYDITE AGGREGATE

Slab No.	Section No.	Water-cement ratio	10-day temperature	Compressive strength: Individual	Compressive strength: Average	Variation from group average
			° F.	Lbs. per sq. in.	Lbs. per sq. in.	Per cent
	2			4,050		7.9
	4			4,320		1.8
3	5	0.88	71.8	4,380	4,398	.4
	6			4,280		2.7
	8			4,960		12.8
	2			4,750		2.3
	4			5,030		3.4
6	5	.76	56.6	4,860	4,864	.1
	6			4,860		.1
	8			4,820		.9
	2			4,730		2.4
	4			5,120		5.6
9	5	.79	55.0	4,385	4,847	9.5
	6			5,390		11.2
	8			4,610		4.9
	2			4,122		.6
	4			4,361		6.4
14	5	.78	64.1	4,343	4,098	6.0
	6			4,407		7.5
	8			3,259		20.5
	2			4,404		5.6
17	4	.78	65.6	5,172	4,665	10.9
	5			4,217		9.6
	6			4,867		4.3
	2			4,673		7.2
	4			5,617		11.5
25	5	.73	38.0	5,090	5,037	1.1
	6			4,693		6.8
	8			5,112		1.5

A study of these data was made in order to draw some conclusion as to the reliability of the indicated strengths for use as a basis upon which to compare the strength values obtained from the other tests.

It will be noted that there is a general relation between the compressive strengths and the water-cement ratio. If all of the values for a particular aggregate are averaged, it will be found that the following relation exists:

Aggregate	Average water-cement ratio	Average compressive strength in pounds per square inch	Average cement factor in bags per cubic yard
Crushed stone	0.91	3,333	5.25
Gravel	.86	3,534	5.11
Haydite	.79	4,652	8.00

The variation in strength of the individual cylinders from the respective group averages was computed, and these data are included in Table 8, as previously noted. From these variations it was found that—

96 per cent of the values show variations of 15 per cent or less;

84 per cent of the values show variations of 10 per cent or less;

75 per cent of the values show variations of 7.5 per cent or less;

59 per cent of the values show variations of 5 per cent or less.

The mean variation for all of the specimens from their respective groups averages is 5.2 per cent.

This analysis indicates that there is no abnormal variation in the strength data for the control cylinders when the number of specimens is considered. It also emphasizes the desirability of having a greater number of specimens than were available in these tests, for an investigation where the close determination of strength is of importance. In spite of careful fabrication and

attention to the details of testing technique, unaccountable strength variations will appear in the data. Occasionally these are of rather large magnitude, but unless there are definite grounds in the history of the specimen for rejecting the data they must be retained and included in the averages. Their effect may be minimized by conducting a large number of tests.

In order to examine the data from the cylinders with respect to effect of temperature, as indicated by the 10-day mean air temperature, the average compressive strengths were arranged in the order of descending temperature values in Table 9. While there appears to be some indication of an increase in compressive strength with an increase in the average air temperature over the 10-day period in the case of the specimens containing crushed stone and gravel aggregates, the relation is not established and there is no such indication in the case of the specimens containing Haydite. The conclusion is drawn, therefore, that the data from the control specimens are normal in character although

TABLE 9.—*Average compressive strength of groups of control cylinders compared with corresponding 10-day mean air temperature*

Gravel aggregate		Crushed-stone aggregate		Haydite aggregate	
Temperature	Strength	Temperature	Strength	Temperature	Strength
° F.	*Lbs. per sq. in.*	*° F.*	*Lbs. per sq. in.*	*° F.*	*Lbs. per sq. in.*
69.2	3,776	64.8	3,300	71.8	4,398
64.7	3,757	64.1	3,153	65.6	4,665
62.7	3,698	62.9	3,865	64.1	4,098
60.9	3,234	60.9	3,570	56.6	4,864
58.3	3,850	54.8	3,090	55.0	4,847
53.2	3,306	52.8	3,672	38.0	5,037
51.6	3,010	47.4	3,051	.....	.....
43.5	3,605	47.4	2,961	.....	.....

TABLE 10.—*Compressive strengths of drilled cores and flexural strengths of sawed beams, expressed as percentages of strengths of corresponding control cylinders*

GRAVEL

Slab No.	Type	A=average compressive strength of control cylinders	Drilled cores; average compressive strength, in pounds per square inch						Sawed beams; average modulus of rupture, in pounds per square inch							
			28 days	Percentage of A	90 days	Percentage of A	1 year	Percentage of A	28 days	Percentage of A	90 days	Percentage of A	180 days	Percentage of A	1 year	Percentage of A
		Lbs. per sq. in.														
12	Hand placed	3,698	4,137	111.9	4,830	130.6	4,885	132.1	.....	.....	701	19.0	616	16.5	580	15.7
23		3,040	3,310	108.9	4,302	141.5	4,775	157.1	427	14.0	554	18.2	476	15.6	547	18.0
	Average	.....	.....	110.4	.....	136.0	.....	144.6	.....	14.0	.....	18.6	.....	16.0	.....	16.8
4	Vibrolithic	3,776	4,760	126.0	5,366	142.1	5,915	156.6	609	16.1	732	19.4	636	16.8	716	19.0
15		3,757	4,893	130.2	5,675	151.0	5,640	150.1	.....	.....	661	17.6	687	18.3	738	19.6
	Average	.....	.....	128.1	.....	146.5	.....	153.4	.....	16.1	.....	18.5	.....	17.6	.....	19.3
7	Electric tamper	3,306	3,975	120.2	4,410	133.4	4,525	136.9	.....	.....	627	19.0	546	16.5	532	16.1
18		3,234	3,961	122.5	4,583	141.7	4,810	148.7	.....	.....	547	16.9	548	16.9	628	19.4
	Average	.....	.....	121.4	.....	137.5	.....	142.8	.....	.....	.....	18.0	.....	16.7	.....	17.8
10	Air hammer	3,850	4,137	107.4	4,920	127.8	5,600	145.4	475	12.3	627	16.3	545	14.2	590	15.3
21		3,605	4,173	115.8	4,780	132.6	5,580	154.8	511	14.2	644	17.9	614	17.0	736	20.4
	Average	.....	.....	111.6	.....	130.2	.....	150.1	.....	13.2	.....	17.1	.....	15.6	.....	17.8

LIMESTONE

13	Hand placed	3,865	3,471	89.8	4,614	119.4	5,150	133.2	.....	.....	669	17.3	423	10.9	523	13.5
24		3,051	3,532	115.8	4,120	135.0	4,900	160.6	461	15.1	606	19.9	506	16.6	691	22.6
	Average	.....	.....	102.8	.....	127.2	.....	146.9	.....	15.1	.....	18.6	.....	13.8	.....	18.0
5	Vibrolithic	3,300	4,701	142.4	4,684	141.9	5,995	181.7	616	18.7	773	23.4	710	21.5	710	21.5
16		3,153	4,652	147.5	5,278	167.4	5,825	184.7	560	17.8	778	24.7	660	20.9	650	20.6
	Average	.....	.....	144.9	.....	154.6	.....	183.2	.....	18.2	.....	24.0	.....	21.2	.....	21.0
8	Electric tamper	3,672	4,444	121.0	5,429	147.8	5,915	160.9	630	17.1	752	20.5	647	17.6	745	20.3
19		3,090	4,210	136.2	4,844	156.8	4,955	160.3	518	16.8	669	21.6	645	20.9	684	22.1
	Average	.....	.....	128.6	.....	152.3	.....	160.6	.....	17.0	.....	21.0	.....	19.2	.....	21.2
11	Air hammer	3,570	4,525	126.8	5,277	147.8	5,440	152.4	495	13.9	731	20.5	680	19.0	720	20.2
22		2,961	3,894	131.5	4,810	162.4	5,485	185.2	501	16.9	622	21.0	619	20.9	678	22.9
	Average	.....	.....	129.2	.....	155.1	.....	168.8	.....	15.1	.....	20.7	.....	19.9	.....	21.6

HAYDITE

3	Hand placed	4,398	3,863	87.8	4,375	99.5	4,880	110.9	401	9.1	469	10.7	196	11.3	305	6.9
14		4,098	3,136	76.5	4,015	98.0	4,185	109.4	496	12.1	718	17.5	372	9.1	264	5.3
	Average	.....	.....	82.2	.....	98.8	.....	110.2	.....	10.6	.....	14.1	.....	10.2	.....	6.1
6	Vibrolithic	4,864	4,014	82.5	4,590	94.4	5,365	110.3	457	9.4	604	12.4	476	9.8	232	4.8
17		4,665	4,165	89.3	4,684	100.4	5,110	109.5	653	14.0	557	11.9	555	11.9	275	5.9
	Average	.....	.....	85.9	.....	97.4	.....	109.9	.....	11.7	.....	12.2	.....	10.8	.....	5.4
9	Electric tamper	4,847	3,555	73.3	4,841	99.9	5,560	114.7	373	11.8	541	11.2	433	8.9	250	5.2
25		5,037	4,091	81.2	4,428	87.9	5,410	107.4	503	10.0	553	11.0	407	8.1	355	7.0
	Average	.....	.....	77.2	.....	93.9	.....	111.0	.....	10.9	.....	11.1	.....	8.5	.....	6.1

somewhat deficient in number, and that the lack of an adequate number of specimens probably accounts for the failure to establish the relation between curing period temperature and compressive strength.

In Table 10 and Figure 9 are shown the compressive and flexural strength data obtained from the various slabs at ages of 1, 3, 6, or 12 months. These data are expressed as percentages of the average compressive strength of the group of five cylinders tested for each of the particular slabs involved. In other words, the strength of this group at 28 days is taken as 100 per cent and all other test values related to it. It should be observed that Table 10 and Figure 9 do not afford a direct comparison of the strength of concretes made with different aggregates. They serve rather to indicate the extent to which the strength characteristics found in the control cylinders were realized in the finished concrete.

It will be noted that when the relative strengths of the concretes containing the three different aggregates are taken into account the trends and relationships shown by Figure 9 are practically the same as those expressed by the strength data before they were modified by the data regarding the control specimens given in Figure 8.

RESULTS OF STRENGTH TESTS SUMMARIZED

The data obtained from tests of flexural and compressive strength indicate that—

1. Mechanical methods of placing resulted in greater flexural and compressive strengths than hand placing in the case of the concretes containing gravel and crushed-stone aggregates.

2. In the case of the concrete containing the Haydite aggregate, mechanical methods of placing produced concrete of higher compressive strength than hand

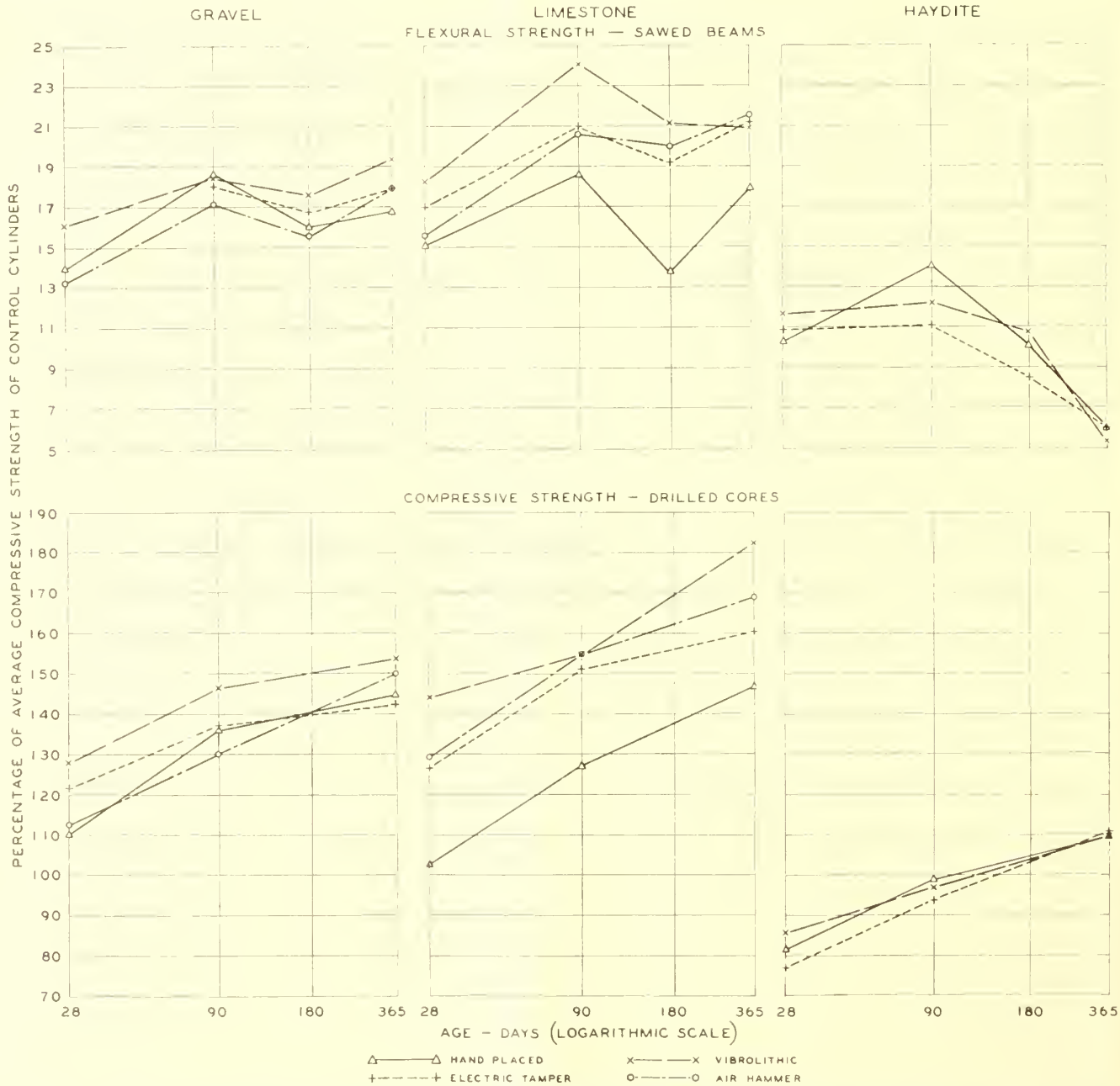


FIGURE 9.—FLEXURAL STRENGTHS OF SAWED BEAMS AND COMPRESSIVE STRENGTHS OF DRILLED CORES, EXPRESSED AS PERCENTAGES OF COMPRESSIVE STRENGTHS OF CORRESPONDING CONTROL CYLINDERS

placing. The effect of vibration on the flexural strength seems to be uncertain, and this may be due to the tendency of the materials to segregate according to their specific gravities when subjected to vibration.

3. The increases in flexural and compressive strengths due to mechanical placing are considerably more marked in the case of the angular aggregate than they are in the case of the rounded aggregate.

4. There is a normal increase in compressive strength with age for all materials and methods of placing.

5. A general increase in flexural strength with age occurred with both the crushed-stone and gravel aggregates, while for some reason not disclosed by these tests a continued retrogression in flexural strength occurred after 90 days in the concrete containing Haydite aggregate.

6. Of the three methods of mechanical placing employed in this investigation, the Vibrolithic process had the greatest effect on strength.

BOND STRENGTH OF EMBEDDED STEEL BARS

In order to develop information concerning the effect of the aggregates and methods of placing on the bond strength of embedded steel, a rather comprehensive

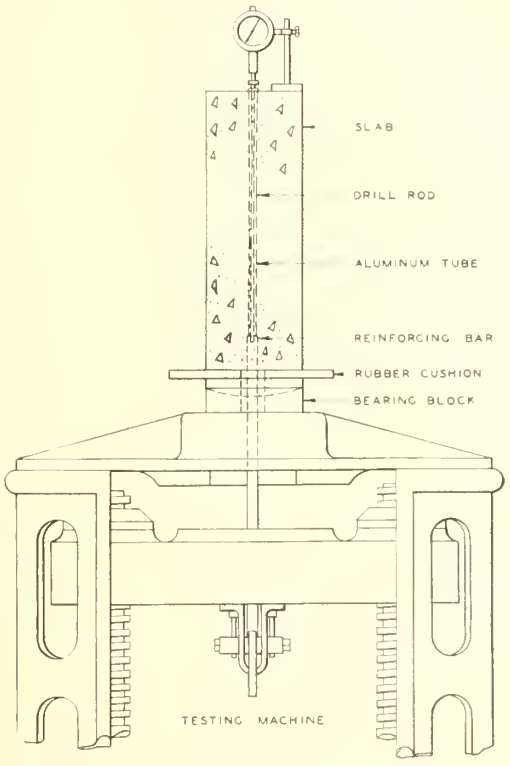


FIGURE 10.—APPARATUS USED IN BOND TESTS OF BARS EMBEDDED HORIZONTALLY

series of bond tests was included in the program. Reinforcing bars of two sizes, each size of both plain and deformed types, were embedded both vertically and horizontally in each of the 11 types of slab. In all, 176 tests were made. The construction of the specimens used in these tests was unique in some respects and for this reason will be described in some detail.

Two-foot lengths of either deformed or plain round bars were embedded in sections 1 and 8 of each of the large slabs. The deformed bars were used in the first series of 11 slabs and the plain bars in the second series, with the exception that deformed bars were used in slabs 23 and 24, and plain bars in slabs 12 and 13. The 1-inch bars were placed in section 1 and the ½-inch bars in section 8 in all cases. The depth of embedment was 6½ inches for both the vertical and horizontal positions.

The vertical bars were passed through holes in the bottom of the slab form and were secured in such a position that the desired embedment was obtained. The upper end was capped with a large cork. After the finishing was completed and the concrete had hardened, the cork was removed and the end of the bar exposed. This permitted a measurement of slip to be made on these specimens.

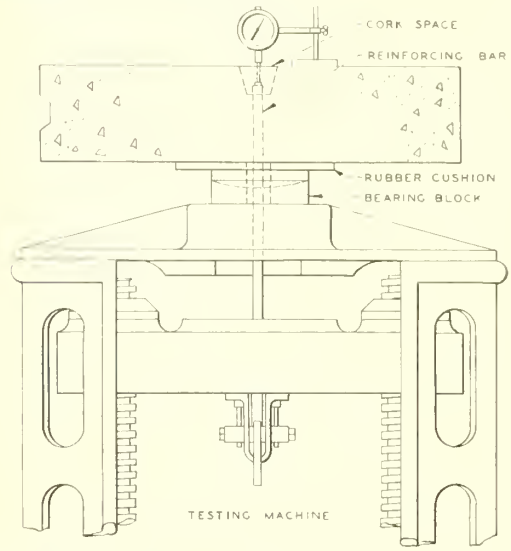


FIGURE 11.—APPARATUS USED IN BOND TESTS OF BARS EMBEDDED VERTICALLY

In the case of the bars placed horizontally a somewhat different scheme was adopted. The embedded end of each bar was drilled axially and tapped to receive a piece of ¼-inch drill rod 18 inches in length. This rod was sheathed with close-fitting tube of thin aluminum, which served to prevent any bond between the concrete and the drill-rod extension. The bars were inserted through horizontal holes in the ends of the slab forms at the mid-depth of the slab. The bar in bond extended into the concrete 6½ inches and the drill-rod extension continued entirely across the remainder of the slab section, the end being supported in a hole in the first transverse wooden separator. This served to hold the bar in position during the placing of the concrete and later, upon removal from the forms, to expose the extension to the embedded end of the reinforcing bar, permitting measurements of slip to be made.

Sections 1 and 8, illustrated in Figure 1, were both divided into two equal parts by a short longitudinal separator, which served to facilitate the handling of the bond specimens.

The tests were made in a universal testing machine, the specimens being supported on a large spherical bearing block provided with a radial hole which allowed the projecting bar to extend through the block. This block was mounted on the fixed head of the testing machine, through which the projecting end of the reinforcing bar passed to be engaged by the grips in the movable head. A rubber cushion was placed between the specimens and the bearing block. During the test the head of the machine was lowered, by power, at a rate of 0.05 inch per minute (idling rate), thus exerting

TABLE 11.—*Unit bond stress at initial slip, in pounds per square inch*

DEFORMED BARS

Slab No.	Aggregate	Method	Horizontal embedment						Vertical embedment					
			1-inch bars			½-inch bars			1-inch bars			½-inch bars		
			Test 1	Test 2	Average	Test 1	Test 2	Average	Test 1	Test 2	Average	Test 1	Test 2	Average
23	Gravel	Hand placed	473	1,310	892	1,133	750	942	996	172	584	568	607	588
4		Vibrolithic	640	1,132	886	730	688	709	1,462	1,082	1,272	872	1,145	1,008
7		Electric tamper	1,190	1,426	1,308	463	255	359	920	1,229	1,074	714	568	641
10		Air hammer	1,318	1,552	1,435	847	759	803	1,352	1,473	1,412	1,117	916	1,016
24	Crushed stone	Hand placed	634	396	515	1,020	954	987	610		610	805	722	764
5		Vibrolithic	1,155	1,369	1,262	478	618	548	1,657	1,141	1,399	901	1,270	1,086
8		Electric tamper	1,562	1,845	1,704	1,165	787	976	1,720	1,888	1,804	922	778	850
11		Air hammer	1,190	1,270	1,230	605	865	735	998	856	927	1,332	984	1,158
3	Haydite	Hand placed	542	457	500	385	255	320	212	402	307	354	1,039	696
6		Vibrolithic	770	1,019	894	322	815	568	1,070	757	914	758	803	794
9		Electric tamper	1,085	1,062	1,074	960	891	926	1,166	768	967	666	1,138	902
PLAIN BARS														
12	Gravel	Hand placed	313	310	312	272	304	288		214	214	550	422	486
15		Vibrolithic	261	108	184	479	213	346	272	356	314	239	316	278
18		Electric tamper	237	275	256		151	151	296	361	328	574	680	627
21		Air hammer	292	351	322	541	277	409	254	300	277	322	381	352
13	Crushed stone	Hand placed	165	82	124	280	196	238	228	280	254	568	470	519
16		Vibrolithic	125	157	141	320	376	348	450	234	342	547	461	504
19		Electric tamper	195	316	256	347	237	292	212	124	168	632	576	604
22		Air hammer	343	230	286	419	391	405	552	375	464	560	601	580
14	Haydite	Hand placed	226	258	242		225	225	251		251	506	387	446
17		Vibrolithic	210	297	254	272	251	262	290	291	290	435	455	445
25		Electric tamper	404	431	418	306	311	308	416	324	370	450	570	510

tension in the bar and tending to withdraw it from the concrete.

The initial and progressive slip of the steel in the concrete was determined by a micrometer dial reading in ten-thousandths of an inch. This dial was supported by a heavy base which rested on the upper surface of the concrete specimen. The stem of the dial rested against the embedded end of the reinforcing bar, or its extension. Figures 10 and 11 show the way in which the bond specimens were set up for test.

The weighing beam of the testing machine was kept in balance as the load was applied and readings of load were taken at the point of initial slip, at the maximum resistance to slip, and at intervals of 0.01 inch slip up to the point where slippage was too rapid to record. As an aid to the accurate determination of the load at the instant of initial slip, a small electric light situated in front of the testing-machine operator was placed in a circuit which was broken by the first movement of the steel bar in the concrete. The operator was thus able to keep the machine balanced and still catch the load reading at the desired instant. It is believed that in all cases the first load reading was obtained before the bar had slipped one ten-thousandth of an inch.

The age of all of the specimens at the time of test was nine months, and during this period they were stored out of doors.

The load in pounds per square inch of bond area at which initial slip occurred is given for each test in Table 11, and these same data are shown graphically in Figure 12.

Figures 13, 14, and 15, show the load-slip curves for all tests with the plain bars. In the tests with the deformed bars, as soon as the bond was broken the bars began to exert a wedging action in the concrete, and failure occurred either by rupture of the bar in tension or by a splitting of the concrete block.

The initial unit bond resistance of the deformed bars is consistently much higher than that of the plain bars as indicated by the values in Table 11. The 1-inch plain bars developed, on the average, only 26 per cent of the resistance of the deformed bars of the same size. The ½-inch plain bars developed 50 per cent of the resistance of the ½-inch deformed bars.

The size of the bar appears to influence the unit bond resistance. In the case of the deformed bars the 1-inch size consistently shows higher bond values than the ½-inch size, while for the plain bars exactly the reverse is true. This is shown in the following summary:

	Lbs. per sq. in. bond stress
Deformed bars:	
1 inch, horizontal embedment	1,064
½ inch, horizontal embedment	716
1 inch, vertical embedment	1,044
½ inch, vertical embedment	864
Plain bars:	
1 inch, horizontal embedment	254
½ inch, horizontal embedment	308
1 inch, vertical embedment	304
½ inch, vertical embedment	486

The data indicate that, on the average, the bars placed in a vertical position developed about 15 per cent more bond resistance than those which were horizontal.

A study was made to determine whether or not the materials from which the concrete was made affected the bond strength. Classifying the data according to materials and averaging all values, the following values are obtained:

Unit bond resistance at initial slip

	Plain bars	Deformed bars
	Lbs. per sq. in.	*Lbs. per sq. in.*
Haydite	335	739
Gravel	321	933
Crushed stone	345	1,035

Thus it would appear that for the low resistance values developed by the plain bars the materials used in the concrete did not have an appreciable effect, but that for high bond-resistance values, such as were developed by the deformed bars, the aggregate does affect the unit bond resistance. It is noted that the variations in these data are in the same sense as the modulus of

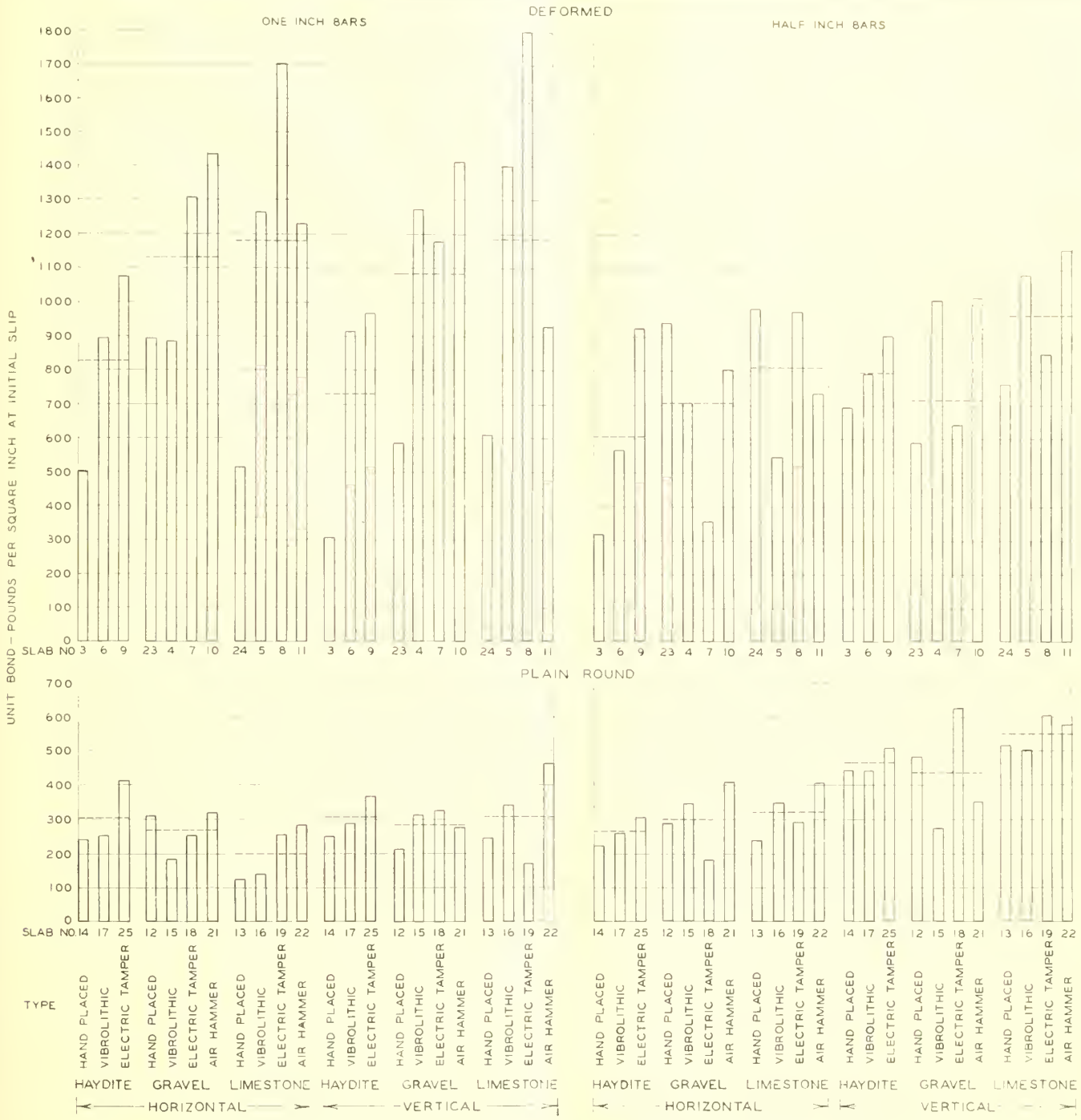


FIGURE 12.—MEAN UNIT BOND VALUES. DASH LINES REPRESENT AVERAGE VALUES FOR EACH GROUP

elasticity values for the three materials although considerably different in degree.

Passing now to a consideration of the effect of the various methods of placing used in this investigation on the initial unit bond resistance, we find that the bond strength of the deformed bars was very noticeably increased by the use of vibratory methods. In the case of plain bars the effect is much less marked. The following summary shows very clearly the effects just mentioned:

Unit bond resistance at initial slip

	Plain bars	Deformed bars
	Lbs. per sq. in.	*Lbs. per sq. in.*
Hand placed	300	642
Vibrolithic	309	945
Electric tamper	357	1,049
Air hammer	387	1,089

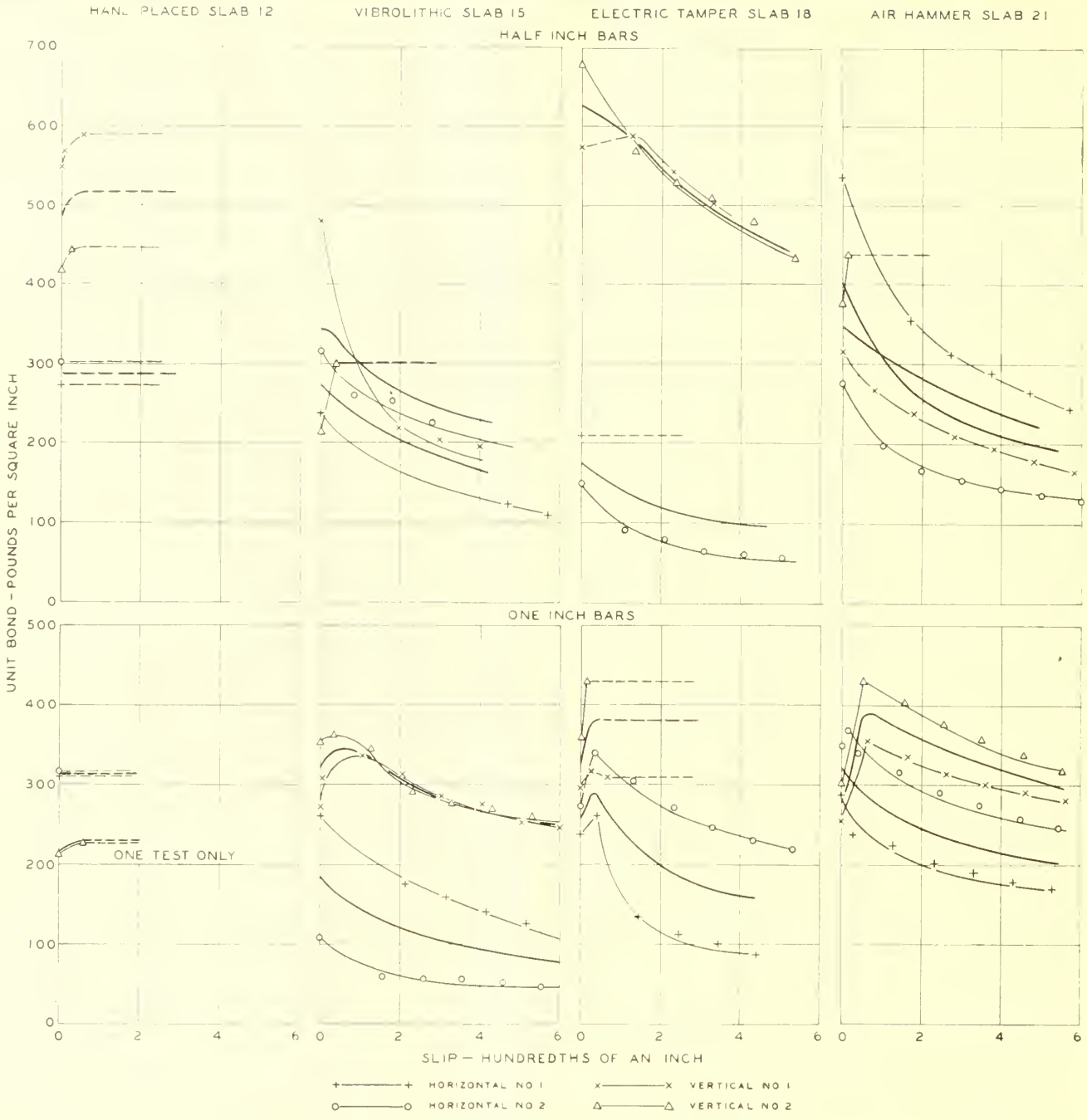


FIGURE 13. LOAD-SLIP CURVES FOR BOND TESTS OF PLAIN ROUND BARS IN GRAVEL CONCRETE

In the tests in which the plain bars were involved the indicated unit bond resistance at the initial slip very probably represents true bond between the steel and the concrete. With deformed bars, however, it is probable that the values measured are a combination of bond and shear.

The wedging action of the deformed bars was so marked that in about four out of every five tests the concrete block was split or the steel bar failed in tension. Of the ten 1-inch bars which were actually pulled out, six were from hand-placed concrete specimens and showed honeycomb around the bar. Of the seven ½-inch bars actually withdrawn, four were from hand-placed concrete showing honeycomb, and all seven of the bars were stressed beyond the elastic limit.

In all of the tests of plain bars the steel was withdrawn without exceeding the elastic limit of the material. It will be noted, in the load-slip curves for these specimens, that in many cases the maximum resistance did not develop at the initial slip but after a very slight movement had occurred. It is probable that this was true in all of the cases, but that the precision of measurement was not sufficient to have it show in the data.

These bond tests indicate that—

1. The apparent unit bond developed by deformed bars is consistently much higher than that developed by plain bars in the same concrete. This comparison is made at the loads at which the first slip was detectable with the apparatus used in these tests.

2. The size of the bar seems to influence the apparent unit bond. With plain bars the ½-inch size shows higher values than the 1-inch size, while with deformed bars the reverse is true.

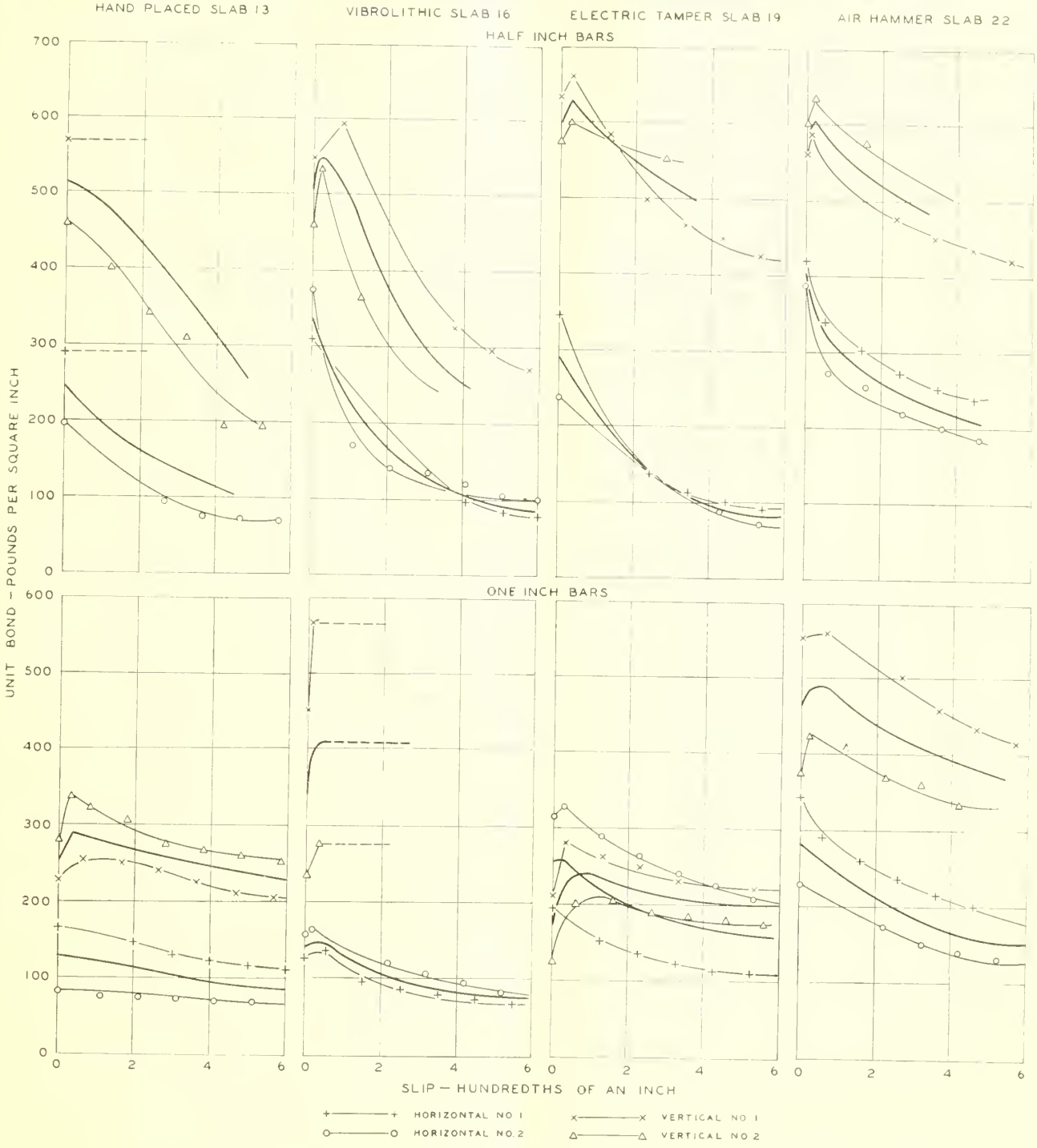


FIGURE 14.—LOAD-SLIP CURVES FOR BOND TESTS OF PLAIN ROUND BARS IN STONE CONCRETE

3. In general, the bars placed in a vertical position developed somewhat greater bond resistance than those placed horizontally.

4. Where slip occurs at comparatively low load, as is true of the plain bars, the aggregate in the concrete does not affect the resistance developed, but where deformed bars are employed the type of aggregate does influence the resistance to a considerable degree.

5. The method of placing used affects the bond resistance, the effect being more marked in the case of the deformed bars than in the case of plain bars. The vibratory methods of placing resulted in better bond than was obtained by the hand-placing method.

TESTS OF THE REINFORCED SECTIONS

As has been noted previously, two sections of each of the large slabs were reinforced with mats of steel bars of the same size and in the same amount as was being considered in connection with the design of floor slabs for the bridge. At the age of 6 months these sections were subjected to bending tests, observations of slab deflection and strain in the concrete being made. The details of the arrangement of the reinforcement in the concrete are shown in Figure 16.

The steel used consisted of ½-inch deformed bars of intermediate grade. The mean cross-sectional area of

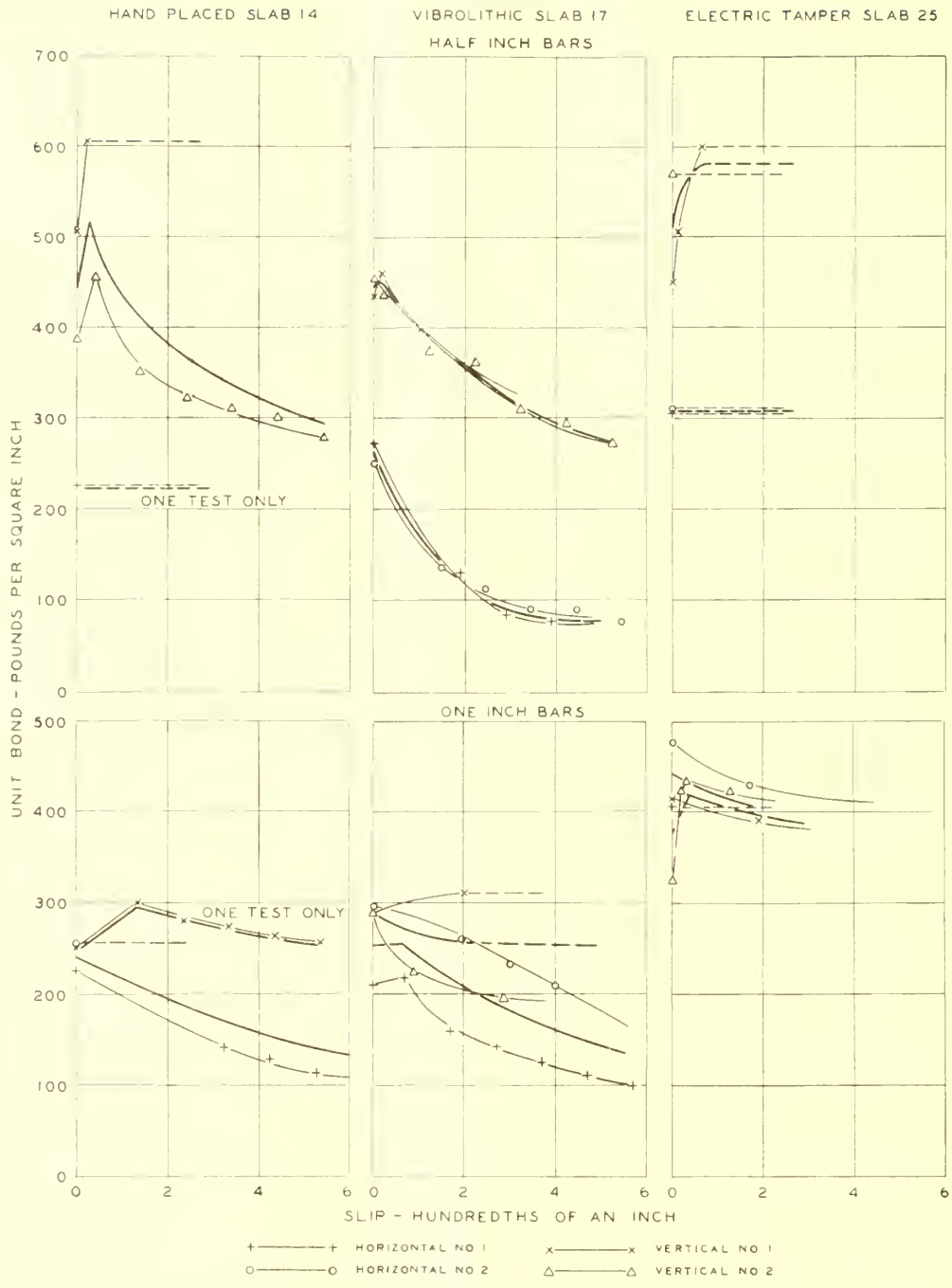


FIGURE 15.- LOAD-SLIP CURVES FOR BOND TESTS OF PLAIN ROUND BARS IN HAYDITE CONCRETE

these bars was measured and found to be 0.191 square inch. The elastic limit averaged 47,355 pounds per square inch (maximum 51,990, minimum 44,240) and the ultimate strength ranged from 84,400 to 72,040 pounds per square inch. The average percentage of elongation was 21.2 per cent. All of the specimens satisfactorily passed the cold bend test.

Figure 17 gives a general view of the way in which the reinforced sections were set up in the testing machine. The weighing table of the machine used is provided with wing extensions, and the slab supports rested on one of these. A 60-inch span was used and the load applied at the third points of this span by the framework shown in the figure just referred to. Horizontal freedom was provided at both load and reaction points by rollers acting against narrow steel plates embedded on the surface of the concrete. There were no particularly unusual features in the test set-up and the details will be omitted.

Deflection of the mid-point of the slab was measured with a micrometer dial reading directly in ten-thousandths of an inch and supported by a stiff tubular steel bridge provided with steel legs which rested on points set in the upper surface of the slab over the reaction supports. One of these legs was provided with a degree of horizontal freedom to take care of temperature and bending displacements. This apparatus is shown in position in the general view, Figure 17.

The deflection data were obtained on both sections from each of the 11 large slabs of the first series and the observations were made after each 5,000-pound increment of load had been applied. These data are given in detail in Table 12, and the load-deflection curves are shown in Figure 18.

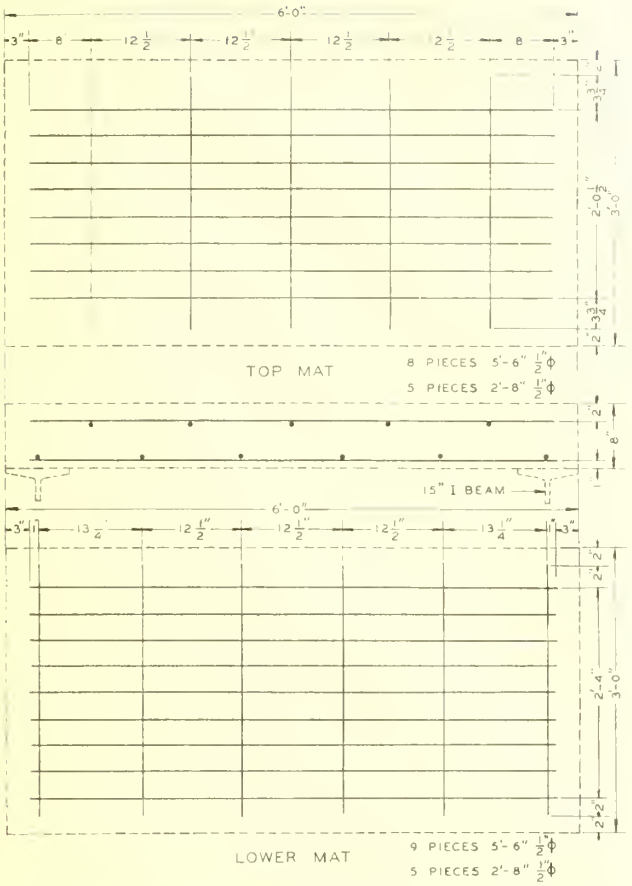

6'-0"
3" 8 12½ 12½ 12½ 12½ 8 3"
TOP MAT
8 PIECES 5'-6" ½" ϕ
5 PIECES 2'-8" ½" ϕ
15" I BEAM
6'-0"
3" 13⅛ 12½" 12½" 12½" 13¼" 3"
LOWER MAT
9 PIECES 5'-6" ⅝" ϕ
5 PIECES 2'-8" ⅝" ϕ

FIGURE 16.—PLACEMENT OF STEEL USED IN REINFORCED SLAB SECTIONS

TABLE 12.—*Detailed record of tests of reinforced concrete slab sections*

SLAB NO. 3, HAYDITE, HAND PLACED; SLAB THICKNESS=8¼ INCHES

Load in pounds	Deflection in inches				Stresses in section 3, in pounds per square inch				
	Section 3	Section 6	Mean	Corrected to 8-inch thickness	Gage No. 1	Gage No. 2	Gage No. 5	Mean	Corrected to 8-inch thickness
5,000	0.0045	0.0044	0.0044	0.0048					
10,000	.0093	.0091	.0092	.0101	210	240	246	232	276
15,000	.0142	.0151	.0146	.0160					
20,000	.0205	.0217	.0211	.0232	486	540	564	530	630
25,000	.0285	.0298	.0292	.0321					
30,000	.0433	.0488	.0460	.0505	1,026	1,026	1,056	1,036	1,234
35,000	.0588	.0648	.0618	.0680					
40,000	.0755	.0806	.0780	.0857	1,506	1,494	1,554	1,518	1,800
45,000	.0890	.0963	.0926	.1018					
50,000	.1127			.1238	1,986	1,980	2,058	2,008	2,390
Load at beam drop, pounds	54,895	50,000	52,448	50,800					

SLAB NO. 4, GRAVEL, VIBROLITHIC; SLAB THICKNESS=8½ INCHES

5,000	0.0015	0.0019	0.0017	0.0021					
10,000	.0025	.0039	.0032	.0040	368	294	250	304	358
15,000	.0051	.0064	.0058	.0072					
20,000	.0067	.0098	.0082	.0102	647	603	544	598	680
25,000	.0121	.0144	.0132	.0165					
30,000	.0192	.0220	.0206	.0258	1,309	1,294	1,145	1,249	1,420
35,000	.0292	.0337	.0314	.0393					
40,000	.0424	.0497	.0460	.0575	2,176	2,205	1,983	2,121	2,410
45,000	.0554	.0625	.0590	.0738					
50,000	.0689	.0764	.0726	.0908	2,881	2,901	2,600	2,794	3,170
55,000	.0828								
Load at beam drop, pounds	59,875	54,390	57,132	53,600					

FIGURE 17.—GENERAL VIEW OF ARRANGEMENT FOR TESTS OF REINFORCED SLAB SECTIONS

TABLE 12.—*Detailed record of tests of reinforced concrete slab sections*—Continued

SLAB NO. 5, LIMESTONE, VIBROLITHIC; SLAB THICKNESS=8⅝ INCHES

Load in pounds	Deflection in inches				Stresses in section 3, in pounds per square inch				
	Section 3	Section 6	Mean	Corrected to 8-inch thickness	Gage No. 1	Gage No. 2	Gage No. 5	Mean	Corrected to 8-inch thickness
5,000	0.0010	0.0015	0.0012	0.0016					
10,000	.0024	.0027	.0026	.0035	180	285	240	235	276
15,000	.0038	.0043	.0040	.0053					
20,000	.0053	.0061	.0057	.0076	390	570	450	470	552
25,000	.0071	.0087	.0079	.0105					
30,000	.0097	.0125	.0111	.0148	630	840	870	780	917
35,000	.0134	.0188	.0161	.0215					
40,000	.0238	.0308	.0273	.0364	1,365	1,505	1,530	1,467	1,730
45,000	.0331	.0462	.0396	.0528					
50,000	.0489	.0601	.0545	.0729					
55,000	.0637	.0748	.0692	.0923					
60,000	.0794	.0888	.0841	.1120					
Load at beam drop, pounds	63,125	63,520	63,322	58,500					

SLAB NO. 6, HAYDITE, VIBROLITHIC; SLAB THICKNESS=8½ INCHES

5,000	0.0038	0.0037	0.0038	0.0046					
10,000	.0078	.0076	.0077	.0094	198	290	205	231	340
15,000	.0113	.0122	.0118	.0144					
20,000	.0163	.0169	.0166	.0202	469	537	528	521	766
25,000	.0224	.0239	.0232	.0283					
30,000	.0328	.0339	.0334	.0407	924	1,022	931	959	1,390
35,000	.0466	.0508	.0487	.0594					
40,000	.0598	.0659	.0628	.0766	1,320	1,365	1,386	1,357	2,000
45,000	.0741	.0837	.0789	.0958					
50,000	.0948	.0997	.0972	.1185	1,789	1,966	2,046	1,934	2,840
Load at beam drop, pounds	54,385	53,975	54,180	47,950					

SLAB NO. 7, GRAVEL, ELECTRIC TAMPER; SLAB THICKNESS=8 INCHES[1]

5,000	0.0025	0.0021	0.0023	0.0023					
10,000	.0052	.0048	.0050	.0050	273	230	301	268	268
15,000	.0087	.0085	.0086	.0086					
20,000	.0152	.0153	.0152	.0152	904	718	1,004	875	875
25,000	.0291	.0265	.0278	.0278					
30,000	.0498	.0421	.0460	.0460	1,995	1,880	2,439	2,105	2,105
35,000	.0688	.0568	.0628	.0628					
40,000	.0886	.0725	.0806	.0806	2,727	2,741	3,300	2,923	2,923
45,000	.1075	.0891	.0983	.0983					
50,000		.1058							
Load at beam drop, pounds	18,260	54,535	51,398	51,398					

SLAB NO. 8, LIMESTONE, ELECTRIC TAMPER; SLAB THICKNESS=8¼ INCHES

5,000	0.0019	0.0019	0.0019	0.0021					
10,000	.0038	.0040	.0039	.0043	287	239	306	277	294
15,000	.0057	.0062	.0060	.0067					

[1] Lower steel displaced in section 3. One bottom bar out, mat raised ½-inch, moved 2 inches toward gages.

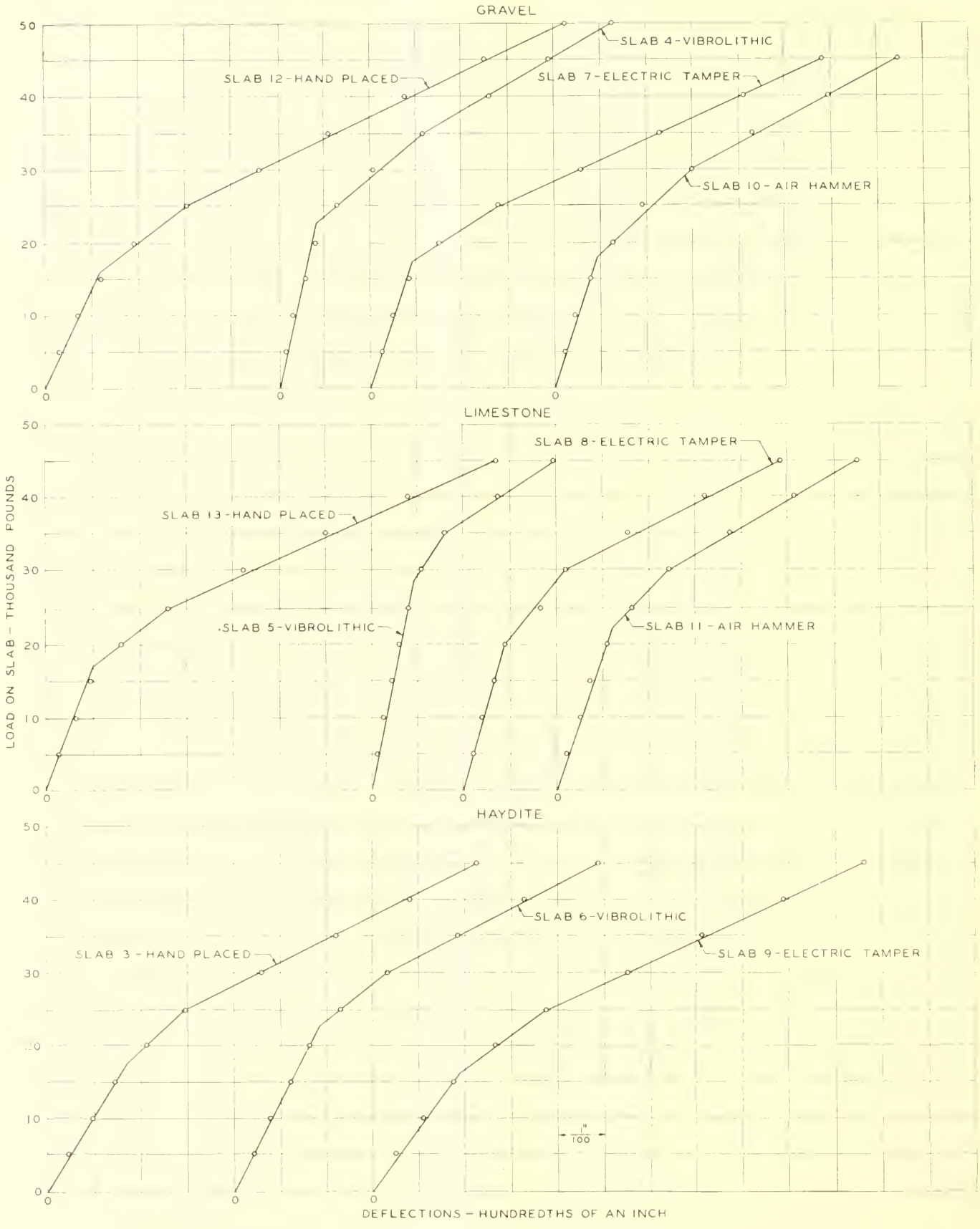


FIGURE 18.—LOAD-DEFLECTION CURVES FOR REINFORCED SLAB SECTIONS

TABLE 12.—*Detailed record of tests of reinforced concrete slab sections*—Continued

SLAB NO. 8, LIMESTONE, ELECTRIC TAMPER; SLAB THICKNESS=8¼ INCHES—Continued

Load in pounds	Deflection in inches				Stresses in section 3, in pounds per square inch				
	Section 3	Section 6	Mean	Corrected to 8-inch thickness	Gage No. 1	Gage No. 2	Gage No. 5	Mean	Corrected to 8-inch thickness
20,000	.0076	.0092	.0084	.0093	734	545	753	677	720
25,000	.0112	.0167	.0140	.0155					
30,000	.0178	.0217	.0198	.0220	1,276	1,343	1,417	1,345	1,430
35,000	.0279	.0360	.0320	.0355					
40,000	.0418	.0519	.0468	.0520	2,233	2,300	2,185	2,239	2,380
45,000	.0552	.0679	.0616	.0684					
50,000	.0695	.0847	.0771	.0856	3,046	3,113	2,997	3,052	3,250
Load at beam drop, pounds	60,015	58,730	59,372	57,500					

SLAB NO. 9, HAYDITE, ELECTRIC TAMPER; SLAB THICKNESS=8 INCHES

5,000	0.0050	0.0051	0.0050	0.0050					
10,000	.0107	.0110	.0108	.0108	306	282	252	280	280
15,000	.0165	.0190	.0178	.0178					
20,000	.0250	.0281	.0266	.0266	630	696	600	642	642
25,000	.0349	.0407	.0378	.0378					
30,000	.0516	.0587	.0552	.0552	1,114	1,236	1,092	1,147	1,147
35,000	.0661	.0760	.0710	.0710					
40,000	.0849	.0933	.0891	.0891	1,576	1,704	1,506	1,595	1,595
45,000	.1009	.1109	.1059	.1059					
Load at beam drop, pounds	49,410	48,730	49,070	49,070					

SLAB NO. 10, GRAVEL, AIR HAMMER; SLAB THICKNESS=8⅛ INCHES

5,000	0.0021	0.0022	0.0022	0.0023					
10,000	.0045	.0051	.0048	.0050	307	256	256	273	281
15,000	.0075	.0086	.0080	.0084					
20,000	.0120	.0130	.0125	.0131	704	589	666	653	674
25,000	.0183	.0192	.0188	.0198					
30,000	.0297	.0301	.0299	.0315	1,395	1,421	1,332	1,383	1,426
35,000	.0442	.0436	.0439	.0462					
40,000	.0597	.0590	.0594	.0625	2,099	2,202	2,074	2,125	2,190
45,000	.0738	.0752	.0745	.0784					
50,000	.0885			.0931	2,829	2,919	2,740	2,829	2,920
55,000	.1040			.1095					
Load at beam drop, pounds	56,675	57,085	56,880	56,000					

SLAB NO. 11, LIMESTONE, AIR HAMMER; SLAB THICKNESS=8 INCHES

5,000	0.0022	0.0024	0.0023	0.0023					
10,000	.0047	.0047	.0047	.0047	279	325	325	310	310
15,000	.0070	.0075	.0072	.0072					
20,000	.0110	.0112	.0111	.0111	759	743	743	748	748
25,000	.0151	.0173	.0162	.0162					
30,000	.0227	.0266	.0246	.0246	1,395	1,394	1,441	1,410	1,410
35,000	.0334	.0417	.0376	.0376					
40,000	.0464	.0559	.0512	.0512	2,309	2,355	2,371	2,345	2,345
45,000	.0587	.0712	.0650	.0650					
50,000	.0724	.0861	.0792	.0792	3,099	3,115	3,161	3,125	3,125
55,000	.0846	.1034	.0940	.0940					
Load at beam drop, pounds	60,590	58,420	59,505	59,500					

SLAB NO. 12, GRAVEL, HAND-PLACED; SLAB THICKNESS=8 INCHES

5,000	0.0030	0.0030	0.0030	0.0030					
10,000	.0068	.0070	.0069	.0069	458	458	397	438	438
15,000	.0114	.0125	.0120	.0120					
20,000	.0172	.0214	.0193	.0193	1,038	1,038	901	992	992
25,000	.0269	.0343	.0306	.0306					
30,000	.0415	.0506	.0460	.0460	1,832	1,816	1,705	1,784	1,784
35,000	.0545	.0674	.0610	.0610					
40,000	.0701	.0844	.0772	.0772	2,626	2,564	2,583	2,591	2,591
45,000	.0868	.1030	.0949	.0949					
50,000	.1038	.1225	.1132	.1132	3,389	3,205	3,285	3,293	3,293
Load at beam drop, pounds	55,040	55,000	55,020	55,020					

SLAB NO. 13, LIMESTONE, HAND-PLACED; SLAB THICKNESS=8 INCHES

5,000	0.0031	0.0028	0.0030	0.0030					
10,000	.0067	.0060	.0064	.0064	295	372	308	325	325
15,000	.0105	.0097	.0101	.0101					
20,000	.0169	.0151	.0160	.0160	796	898	886	860	860
25,000	.0283	.0238	.0260	.0260					
30,000	.0427	.0418	.0422	.0422	1,579	1,694	1,554	1,609	1,609
35,000	.0581	.0612	.0596	.0596					
40,000	.0731	.0827	.0779	.0779	2,208	2,375	2,260	2,281	2,281
45,000	.0873	.1061	.0967	.0967					
Load at beam drop, pounds	53,990	47,320	50,655	50,665					

Strain measurements were made in one of the two reinforced sections from each of the 11 large slabs of the first series and the observations were made after each 10,000-pound increment of load had been applied. Strains in the concrete were measured at five points in the upper surface of the slab, the location of which is shown in Figure 19. These measurements were made with a hand strain gage between brass points set 8 inches apart. Corrections were made for temperature changes in the gage. The data obtained are given in detail in Table 12 and shown graphically in Figure 20 where the average stress across the transverse axis of the slab is plotted as a function of the total load. The observed strains at the other two gage lengths along the longitudinal axis of the slab were used as a check on the distribution of stress in the middle third of the slab.

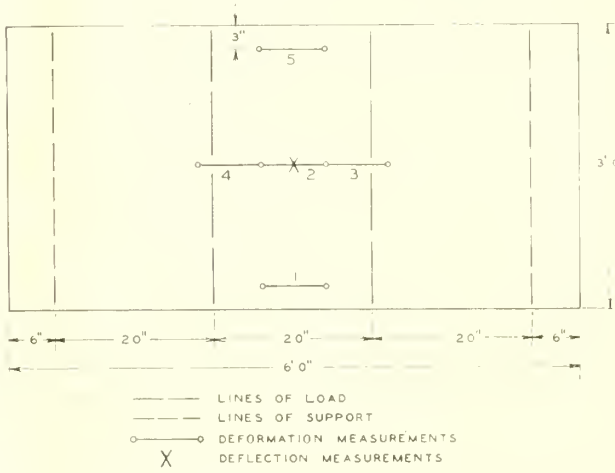


FIGURE 19.—GAGE LAYOUT ON REINFORCED SLAB SECTIONS

In converting the observed strains into the stress values given in Table 12, each value was multiplied by the modulus of elasticity of the concrete for that particular slab, as shown by the tests made at the 90-day age. These values of modulus of elasticity are very probably a few per cent lower at 90 days than they were at the time the reinforced sections were tested. Inspection of the data on the effect of age on the modulus of elasticity indicates that this difference would be small. (See Table 7.)

In Table 13 certain load and strength data for the reinforced slab sections have been assembled for purposes of comparison. The first two columns give the load on the slab at the time when the load-stress and and load-deflection curves respectively change in slope. It is realized that there are not a sufficient number of observations to fix this value very accurately, and the values given in the table were determined by projecting the initial and final slopes to their point of intersection. It is quite possible that the change in slope was gradual and that this change began at a lower load than is tabulated.

Column 3 contains the modulus of rupture of the concrete as determined by the tests of the sawed beams at the age of 6 months (the same age at test as the reinforced slabs). Columns 4, 5, and 6 contain stress values (in the concrete) arrived at in various ways. These values were computed in each case for the load A (Table 13) which is the load corresponding to the change in slope of the load-stress curve. Columns 4 and 5 contain stress values computed for the lower and upper surfaces of the concrete in the middle third of the slab. These values were

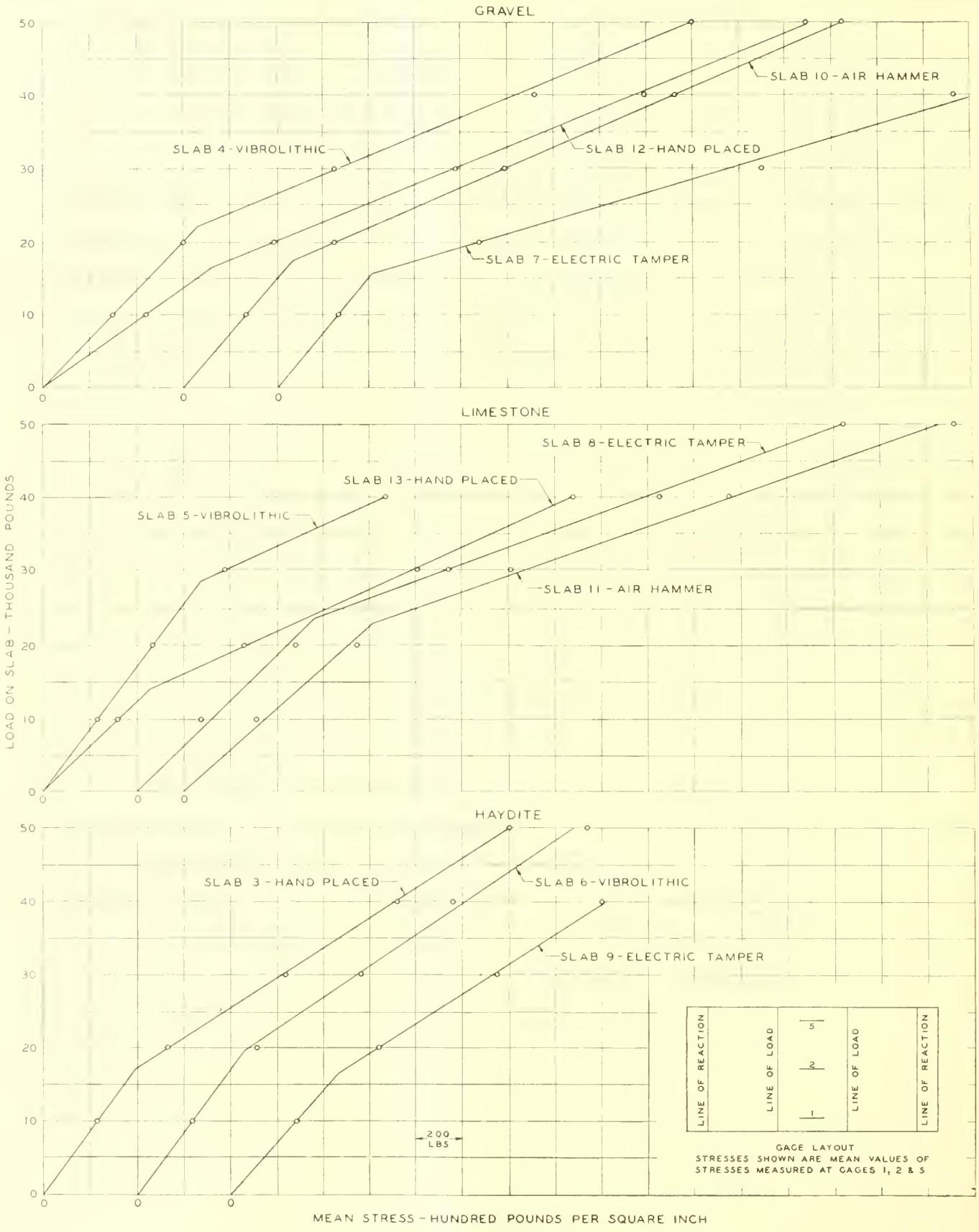

GRAVEL
SLAB 4 - VIBROLITHIC
SLAB 10 - AIR HAMMER
SLAB 12 - HAND PLACED
SLAB 7 - ELECTRIC TAMPER
LIMESTONE
SLAB 8 - ELECTRIC TAMPER
SLAB 13 - HAND PLACED
SLAB 5 - VIBROLITHIC
SLAB 11 - AIR HAMMER
HAYDITE
SLAB 3 - HAND PLACED
SLAB 6 - VIBROLITHIC
SLAB 9 - ELECTRIC TAMPER
200 LBS
LOAD ON SLAB - THOUSAND POUNDS
MEAN STRESS - HUNDRED POUNDS PER SQUARE INCH
LINE OF REACTION
LINE OF LOAD
LINE OF LOAD
LINE OF REACTION
GAGE LAYOUT
STRESSES SHOWN ARE MEAN VALUES OF STRESSES MEASURED AT GAGES 1, 2 & 5

FIGURE 20.—LOAD-STRESS CURVES FOR REINFORCED SLAB SECTIONS

TABLE 13.—*Comparative load and stress data for reinforced sections*

Method of placing	Slab No.	Load in pounds at change of slope: Load-stress curve (load A)	Load in pounds at change of slope: Load-deflection curve	Modulus of rupture of sawed beams, in pounds per square inch, age 6 months	Computed stress in pounds per square inch for load A: Bottom of slab	Computed stress in pounds per square inch for load A: Top of slab	Strain [1] × modulus of elasticity [2] $s=\delta E$
		(1)	(2)	(3)	(4)	(5)	(6)
HAYDITE CONCRETE							
Hand placed	3	17,500	18,000	496	486	485	410
Vibrolithic process	6	19,500	22,000	476	518	515	450
Electric tamper	9	16,250	16,500	433	458	462	450
LIMESTONE CONCRETE							
Hand placed	13	13,750	16,500	423	461	435	440
Vibrolithic process	5	28,250	27,500	710	817	803	660
Electric tamper	8	22,250	20,000	647	711	693	700
Air hammer	11	21,500	22,500	680	714	690	750
GRAVEL CONCRETE							
Hand placed	12	16,250	17,000	610	548	521	700
Vibrolithic process	4	22,000	22,500	636	662	643	660
Electric tamper	7	16,000	17,500	546	554	519	440
Air hammer	10	18,000	18,500	545	574	552	490

[1] Strain is the average of 3 gages on transverse axis of slab.
[2] Average modulus of elasticity from 90-day tests of cores from this same slab.

computed by the usual flexure formula, taking into account the actual dimensions of the slab and the amount and position of the longitudinal steel present in it. The ratio between the moduli of elasticity of the steel and of the concrete was computed on the basis of a value of 30,000,000 pounds per square inch for the steel and the value shown by the 90-day core tests for the concrete. Column 6 was computed from the measured strains in the concrete in the upper surface of the slab and the modulus of elasticity determined for the particular slab in question from the 90-day cores.

In Figure 21 a comparison is made of the relation between the various slabs as to—

(*a*) Load at which a change of slope occurs in the load-stress curve, and

(*b*) Modulus of rupture of the concrete at the same age.

A study of the data in Table 13 and Figure 21 clearly indicates that the change in slope of the load-stress curve is caused by the failure of the concrete in flexure. Considering the small number of tests which are represented by these data, it is believed that they are remarkably concordant on this point.

Since it is indicated that the first change in the structural behavior of the reinforced slabs occurred when the concrete failed in tension on the lower side of the slab, it will be of interest to examine the data to see what took place after the concrete ceased to bear its proportionate share of the tensile stress. The data show that after the first change in slope of the load-stress curve the structure continued to act elastically up to a certain point, which was indicated by the dropping of the weighing beam of the testing machine. In Table 14 the load on the various slabs at the time this elastic failure occurred are noted, together with the computed tensile stresses in the steel at these loads. In computing these values the dimensions of both the steel and concrete for the particular slab under consideration were used.

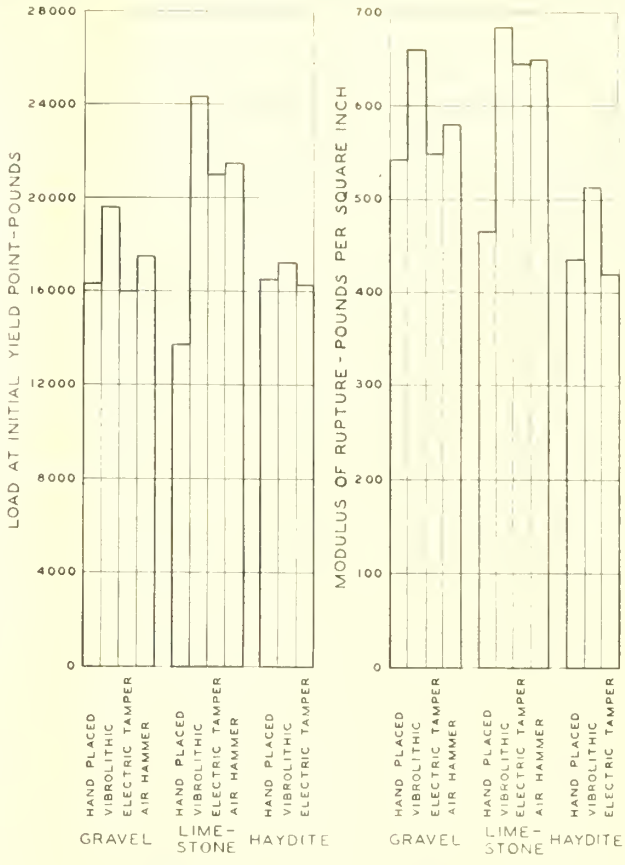


FIGURE 21.—COMPARISON OF VALUES OF MODULUS OF RUPTURE OF PLAIN CONCRETE BEAMS AND VALUES OF LOAD ON REINFORCED CONCRETE SLABS AT INITIAL YIELD POINT. IN LEFT-HAND PANEL ARE GIVEN VALUES OF LOAD ON REINFORCED SLABS AT CHANGE IN SLOPE OF LOAD-STRESS CURVES, CORRECTED TO 8-INCH THICKNESS. VALUES OF MODULUS OF RUPTURE OF SAWED BEAMS AT 180 DAYS, GIVEN IN RIGHT-HAND PANEL, ARE MEANS OF 4 TESTS ON GRAVEL CONCRETE, AND 8 TESTS ON LIMESTONE AND HAYDITE CONCRETE

TABLE 14.—*Loads on reinforced slabs at failure and corresponding stresses in steel*

Slab No.	Section No.	Aggregate	Method of placing	Total load	Tensile stress in steel
				Pounds	*Pounds per square inch*
3	3	Haydite	Hand placed	54,895	48,400
3	6	do	do	50,000	44,200
6	3	do	Vibrolithic process	54,385	46,000
6	6	do	do	53,975	45,600
9	3	do	Electric tamper	49,410	46,000
9	6	do	do	48,730	45,300
13	3	Limestone	Hand placed	53,990	48,200
13	6	do	do	47,320	44,000
5	3	do	Vibrolithic process	63,125	49,000
5	6	do	do	63,520	49,800
8	3	do	Electric tamper	60,015	50,000
8	6	do	do	58,730	50,000
11	3	do	Air hammer	60,590	52,200
11	6	do	do	58,420	50,500
12	3	Gravel	Hand placed	55,040	47,600
12	6	do	do	55,000	47,600
4	3	do	Vibrolithic process	59,875	48,500
4	6	do	do	54,390	44,100
7	3	do	Electric tamper	48,260	50,000
7	6	do	do	54,535	46,500
10	3	do	Air hammer	56,675	49,100
10	6	do	do	57,085	50,600

It is evident from this tabulation that the second change in structural behavior occurred as a result of the steel being stressed to the yield point. It is recalled that the yield point of the reinforcement was found to range from 44,240 to 51,990 pounds per square inch.

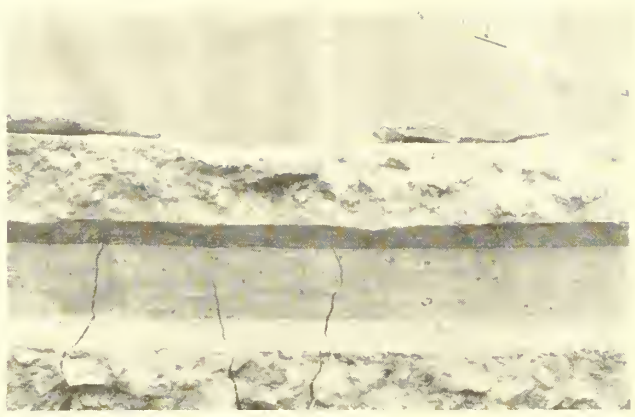


FIGURE 22.—FAILURE OF A REINFORCED SLAB. NOTE WIDE CRACKS REMAINING AFTER REMOVAL OF LOAD AND SPALLING OF CONCRETE ON COMPRESSION SURFACE OF SLAB

Thus it may be concluded that the ultimate failure of the reinforced slabs was due to the failure of the steel in tension. The appearance of the slab specimens also indicated this to be true. Figure 22 shows the appearance of the side of one of these slabs after failure. This particular slab spalled at the upper surface, as shown in the photograph. This spalling occurred just before final failure.

ABSORPTION TESTS

An effort was made to determine the relative ability of the concrete from the various slabs to absorb water. For this purpose two specimens 8 inches square and approximately 18 inches in length were selected from the broken sawed-beam specimens from each of the large slabs. These specimens were air-dried in a sheet-iron building for several weeks, until all of them had ceased to change in weight. The concrete was from 10 to 12 months old at the time that this drying-out period was completed. When all of the specimens had reached a condition of constant weight in the dry air of the building, they were immersed in a tank of water (at air temperature). After periods of immersion of 1, 2, 8, 28, 56, and 140 days they were removed from the water; the surface moisture was blotted off with a damp cloth; and the specimens were weighed.

The progressive absorption of the various concretes is given in Table 15. Average values for four specimens are tabulated, and these data are also shown graphically in Figure 23. In this figure it will be noted that for each aggregate the methods of placing have been arranged from left to right in the order of ascending values.

From these data it would appear that the vibratory methods of placing produced a perceptibly less absorptive concrete than was obtained by hand placing. This difference was most marked with the concrete containing the Haydite aggregate and least with the concrete containing the gravel aggregate. Of the three vibratory methods, the Vibrolithic method of placing appears to be the most effective from the standpoint of reducing absorption. These absorption values appear to be, in a general way, a measure of the density of the concrete as indicated by the unit weight values given in Table 3. For each aggregate the highest absorption and lowest weight per cubic foot are found to relate to hand-placed concrete, while the lowest absorption and highest unit weight are found to be for concrete

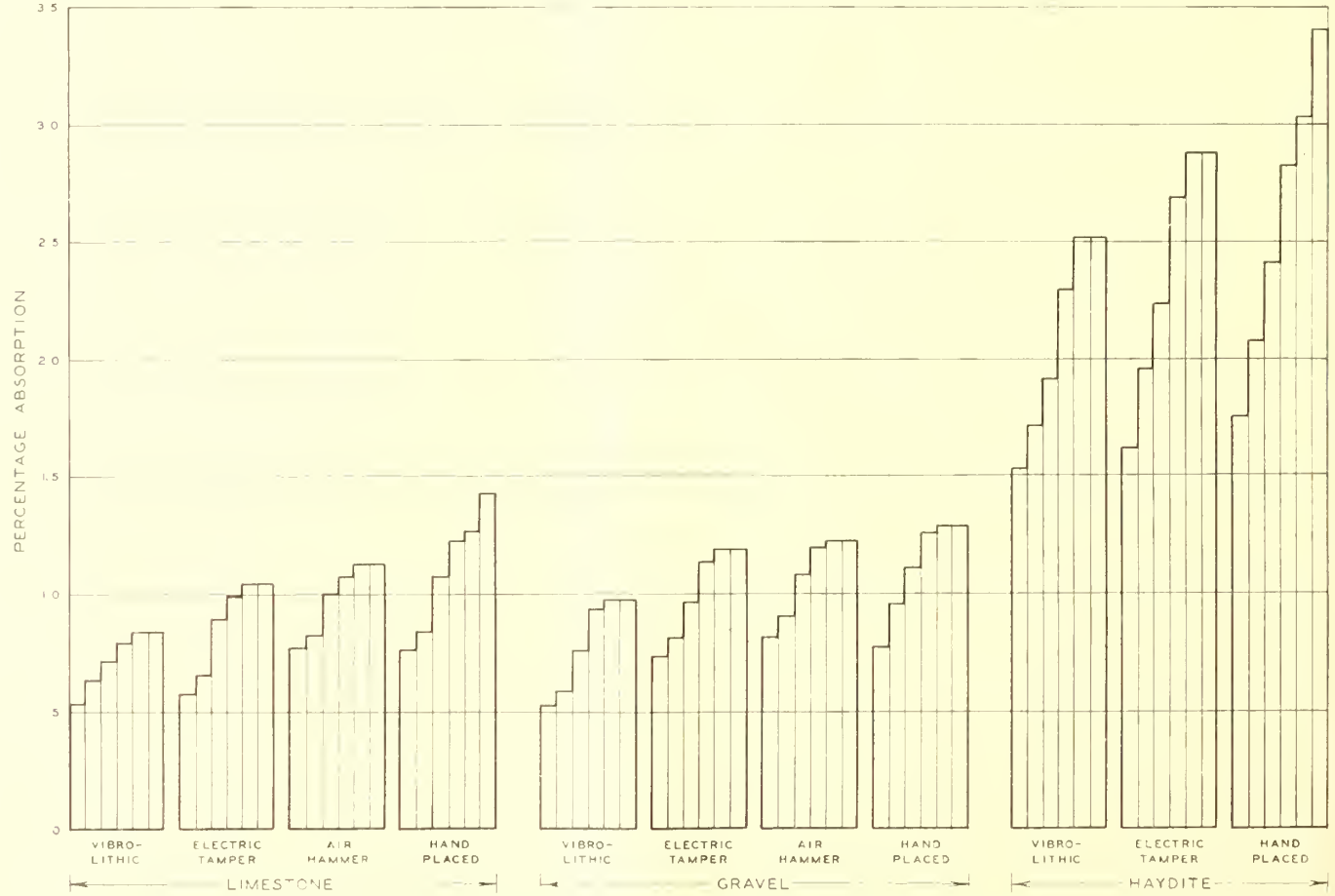


FIGURE 23.—EFFECTS OF METHOD OF PLACING AND TYPE OF AGGREGATE ON PROGRESSIVE ABSORPTION OF CONCRETE SPECIMENS AT 1, 2, 8, 28, 56, AND 140 DAYS' IMMERSION. EACH VALUE GIVEN IS THE MEAN OF FOUR TESTS

TABLE 15.—*Percentage of absorption (based on dry weight) of 8 by 8 by 18 inch concrete specimens—Values given are averages of determinations on four specimens*

LIMESTONE AGGREGATE

Method of placing	Period of immersion (in days)					
	1	2	8	28	56	140
Vibrolithic process	0.53	0.64	0.72	0.80	0.84	0.84
Electric tamper	.58	.65	.89	1.00	1.05	1.05
Air hammer	.78	.83	1.00	1.08	1.13	1.13
Hand placed	.76	.84	1.07	1.23	1.27	1.43

GRAVEL AGGREGATE

Vibrolithic process	0.53	0.59	0.76	0.94	0.98	0.98
Electric tamper	.74	.82	.97	1.14	1.20	1.20
Air hammer	.81	.90	1.08	1.20	1.24	1.24
Hand placed	.78	.96	1.12	1.27	1.29	1.29

HAYDITE AGGREGATE

Vibrolithic process	1.53	1.71	1.94	2.30	2.51	2.51
Electric tamper	1.62	1.97	2.24	2.69	2.88	2.88
Hand placed	1.77	2.08	2.41	2.84	3.02	3.40

placed by the Vibrolithic method. The unit weight of the concrete containing gravel aggregate is about the same as that of the concrete containing the limestone and much higher than that containing Haydite, while from the standpoint of absorption the inverse relation is again found to be generally true.

RESISTANCE TO WEAR

The purpose for which these concrete slabs were to be used made the matter of resistance to surface wear of considerable interest. Tests were made on the 11 large slabs of the second series to determine the relative surface hardness or resistance to wear, using a special testing device developed by the bureau for this purpose. This apparatus is described in detail in a former issue of Public Roads[2] and consists of three narrow wheels of hardened steel set tangentially on a circular plate which is rotated about its center under a constant load at a constant speed of 35 revolutions per minute. The path followed by the wheels is a circle 21 inches in diameter.

These wheels follow each other around this path and soon begin to wear a groove. The rate at which the depth of this groove develops is determined by measurements with a micrometer dial after some definite number of revolutions of the plate and is taken as an indication of the relative surface hardness of the concrete.

As said before, this test was made on each of the 11 slabs of the second series. Two tests were made on each slab, and the age of the concrete at the time of test was 10 months.

The data obtained are shown graphically in Figure 24.

From these data it is evident that for all three aggregates the use of vibratory methods of placing tended to decrease the surface hardness of the concrete and that this effect was particularly pronounced in the case of the concrete containing the Haydite aggregate. The only exception is in the case of the concrete containing the gravel aggregate and placed by the Vibrolithic method. The concrete containing gravel aggregate which was placed by the two other vibrating methods consistently showed a softer surface than that which was

[2] Public Roads, vol. 10, No. 5, July, 1929, A Test for Indicating the Surface Hardness of Concrete Pavements.

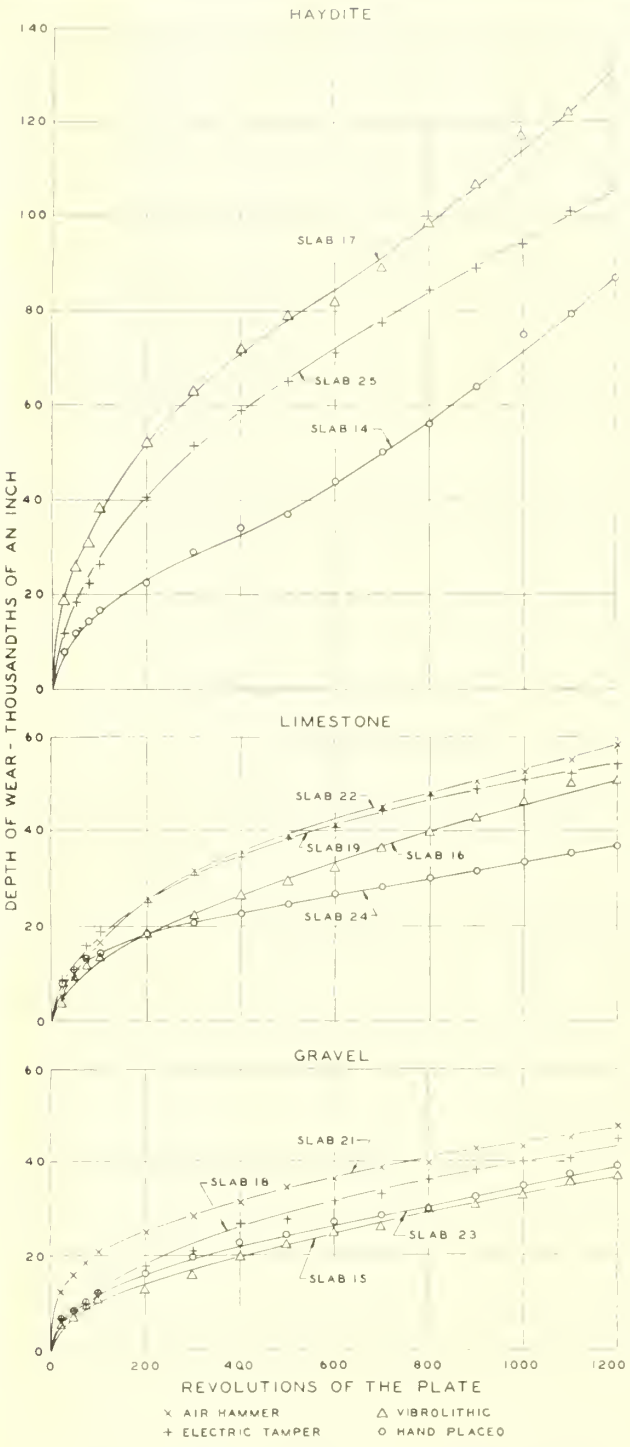


FIGURE 24.—RESULTS OF WEAR TESTS; AVERAGE CURVES DETERMINED BY TWO TESTS ON EACH SLAB

placed by the hand method. It is believed that this is due to the fact that the vibration tends to bring the water to the surface and this water carries with it the finer, less resistant particles. The result is that there is an excess of this material on or near the surface of the slab as the water evaporates.

Comparison may also be made between the different aggregates, and it will be noted that the depth of wear was approximately twice as great on the concrete containing Haydite as on the other aggregates.

The addition of the trap rock to the surface of the slabs placed by the Vibrolithic process had for its

purpose the protection of the surface from wear through the wear resistance of the stone. Presumably, when the mortar skin of the concrete becomes worn the layer of trap rock is exposed and assumes the burden of wear resistance.

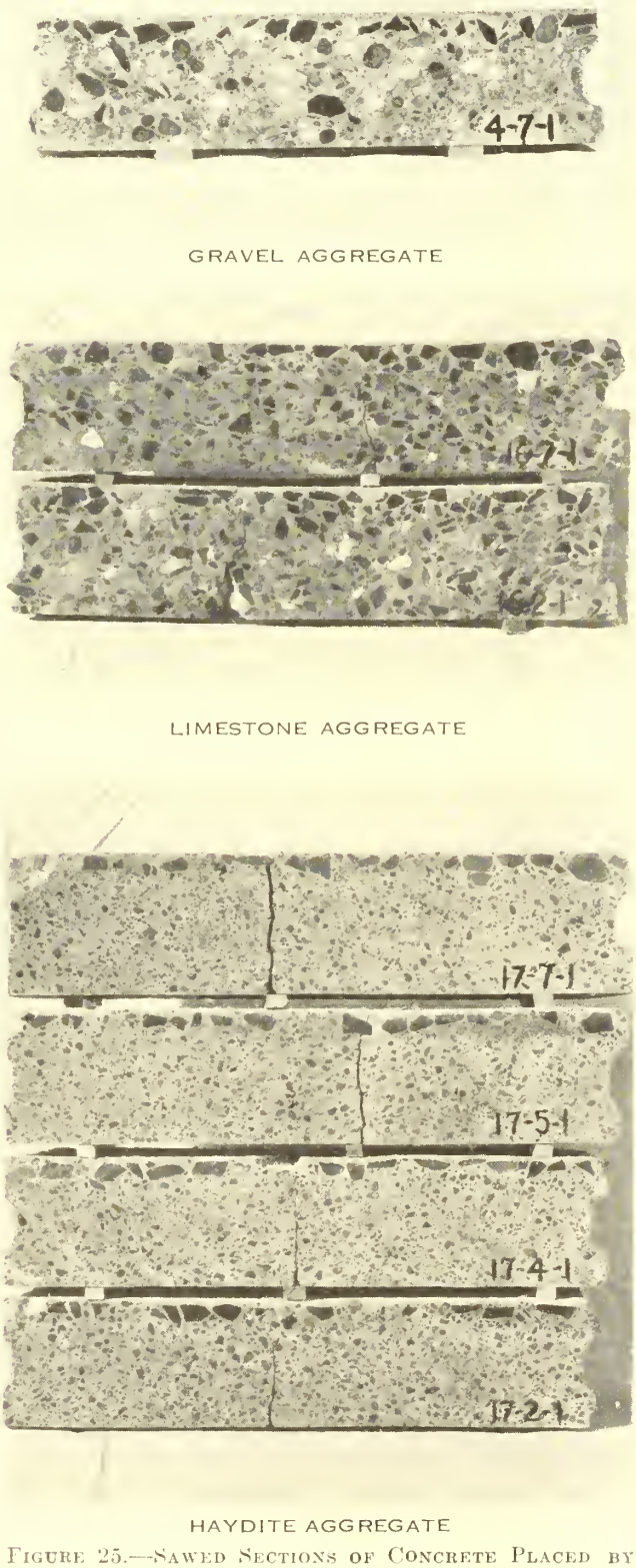


FIGURE 25.—SAWED SECTIONS OF CONCRETE PLACED BY VIBROLITHIC PROCESS, SHOWING FINAL POSITION OF TRAP ROCK ADDED DURING FINISHING

The sawed beams furnished an unusual opportunity to study the position of these fragments of trap rock in the hardened concrete and to judge to what degree protection to the surface is afforded by them. Figure 25 shows typical faces of the sawed sections for all three types of aggregate. The pieces of stone in question can be identified without difficulty near the upper surface of all of the specimens. It is quite apparent that, so far as these slabs are concerned, the trap rock would be of little or no value as a surface wearing course.

RESISTANCE TO REPEATED FREEZING

Since durability is one of the most important qualities to be sought in exposed concrete work, an effort was made to obtain information regarding the relative resistance to weathering of the various concretes under test by subjecting specimens from each of the large slabs to alternate freezing and thawing.

Specimens 8 by 8 by 4 inches in size were sawed from portions of the sawed beams used in the flexure tests, one from section 2 and one from section 7 of each large slab, a total of 44 specimens. Four of the six faces of each specimen were sawed surfaces, one was the finished (or upper) slab surface, and the other was the surface left by the wood form on which the slab was cast. The specimens were separated into two groups of 22 each, one comprising the blocks cut from section 2 of each large slab, the other those from section 7. These two groups were alternated in the freezing cycle, one being frozen while the other was being thawed.

All of the specimens were subjected to 100 cycles of alternate freezing and thawing. Freezing was accomplished in a small compression refrigerator built especially for this work. The specimens were placed in the freezing chamber in a saturated condition and allowed to remain for 24 hours. Periodically thermograph records were made of the air temperature in the chamber and of the concrete temperature at the center of one of the specimens.

These charts showed that the minimum temperature of the concrete ranged from $-10°$ to $-15°$ C. over the 8-month period of the tests and that this minimum temperature was maintained for from 14 to 17 hours of each 24-hour cycle of freezing. These variations were caused by the variation in the temperature of the specimens when placed in the freezing chamber.

At the end of the 24-hour freezing period the specimens were removed from the refrigerator and immersed in a tank of water at room temperature, where they were allowed to remain for 24 hours.

All specimens were examined carefully at frequent intervals and any changes in appearance noted. The detailed observations for each specimen are given in Table 16, and photographs of all specimens are grouped in Figures 26, 27, and 28.

An examination of the specimens after the completion of 100 alternations of freezing and thawing leads to the conclusion that all of the specimens are in very good condition after a rather severe test. In general, the concrete is all sound and the sawed surfaces are practically intact. Differences in the resistance of the various groups of specimens can be detected, however, and these permit some comparison to be made of the effect on resistance of the aggregates and of the methods of placing. It will be noted that many of the observed effects of freezing listed in Table 16 are not of a serious nature so far as the mass of the concrete in the specimen is concerned.

Studying the data from the standpoint of the aggregates used, it is apparent that the concrete containing the siliceous gravel was affected least by the repeated freezings. This concrete was practically unchanged by the test in so far as could be determined by a careful examination of its physical appearance. The concrete containing the limestone coarse aggregate was more difficult to consolidate because of the angularity of the stone fragments. The water-cement ratio was higher than that of the concrete in which gravel was used and more void spaces are to be found around the coarse aggregate particles. The result was that freezing did considerably more damage to the specimens containing the limestone than to those containing the siliceous gravel. The void spaces proved to be particularly vulnerable to freezing attack.

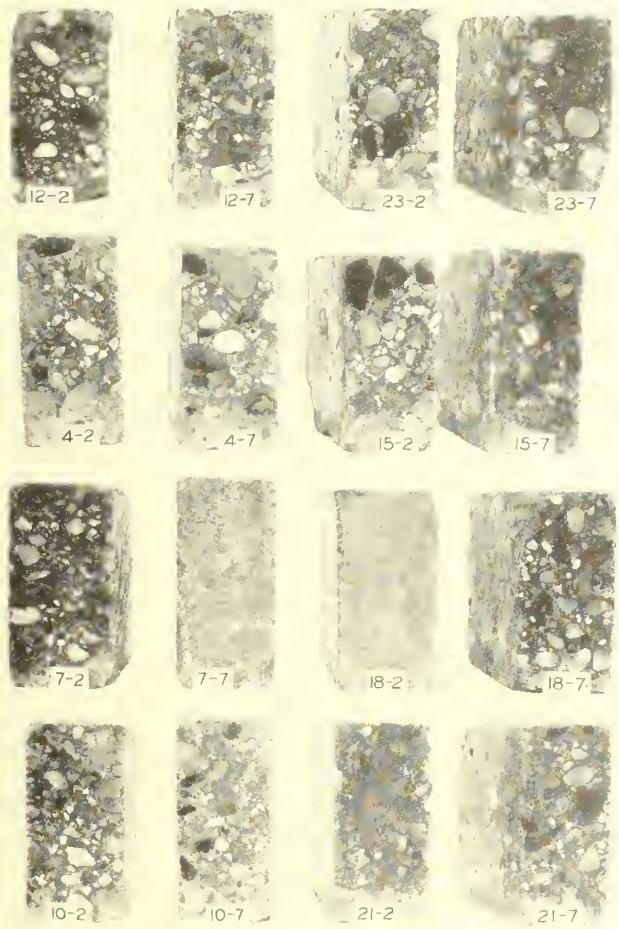


FIGURE 26.—RESISTANCE OF GRAVEL CONCRETE SPECIMENS TO REPEATED FREEZING. SLAB AND SECTION NUMBERS ARE GIVEN ON THE PHOTOGRAPHS. SLABS 12 AND 23, HAND PLACED; 4 AND 15, VIBROLITHIC; 7 AND 18, ELECTRIC TAMPER; 10 AND 21, AIR HAMMER

In only one instance was any disintegration of the limestone aggregate noted, the weakness apparently being in the mortar around the void spaces and on the finished surface which was originally the top of the slab. This indicates that the greater damage noted in the specimens containing the crushed-stone aggregate was possibly due to the fact that a higher water-cement ratio was required for workability rather than to the fact that limestone was used. All of the specimens containing the Haydite aggregate showed isolated

TABLE 16.—*Detailed observations of the effect of repeated freezing*

Type	Slab No.	Sec. No.	Effects observed
GRAVEL AGGREGATE			
Hand placed	12	2	All specimens unchanged after 100 alternations.
		7	
Do	23	2	
		7	
Vibrolithic process	4	2	Do.
		7	
Do	15	2	
		7	
Electric vibrator	7	2	None.
		7	Slight scaling on finished face (toward the front of the photograph) noted after 80 alternations.
Do	18	2	None.
		7	Do.
Air Hammer	10	2	Do.
		7	3 small pieces of coarse aggregate out of sawed surface after 80 alternations.
Do	21	2	None.
		7	Do.
LIMESTONE AGGREGATE			
Hand placed	13	2	Some crumbling of mortar toward a honeycombed area after 61 alternations.
		7	Some crumbling of mortar around a honeycombed area after 80 alternations.
Do	24	2	The lower third of this specimen was badly honeycombed, and after 59 alternations the mortar began to crumble. Considerable disintegration noted after 72 alternations.
		7	Some crumbling of mortar at 1 small spot on a sawed face noted after 80 alternations. No change after 100 alternations.
Vibrolithic process	5	2	Slight softening of the mortar above the trap rock on the finished surface apparent after 80 alternations.
		7	1 piece of limestone 1½ inches in diameter disintegrated after 72 alternations.
Do	16	2	Slight softening of finished surface noticeable after 80 alternations.
		7	Section of mortar on finished face, above the trap rock, loosened after 80 alternations.
Electric vibrator	8	2	Finished surface softened to a depth of 1/16 inch after 80 alternations.
		7	None
Do	19	2	Finished surface softened to a depth of 1/16 inch. First noted after 80 alternations (to the front of photograph).
		7	Finished surface softened to a depth of 1/16 inch. First noted after 80 alternations.
Air hammer	11	2	Finished surface showed scale after 61 alternations, and 1 corner, where honeycombed, crumbled after 80 alternations.
		7	Mortar on finished surface softened. Apparent after 72 alternations.
Do	22	2	Do.
		7	Mortar on 1 sawed face noticeably softened after 80 alternations. The same effect noted on 1 part of the finished surface.
HAYDITE AGGREGATE			
Hand placed	3	2	Slight scale on finished surface. First apparent after 80 alternations. Also certain particles of the coarse aggregate appeared to soften under repeated freezing. After 30 alternations it was noted that the nonporous particles, which apparently were not completely fused in manufacture, began to soften.
		7	Condition the same as section 2.
Do	14	2	Some crumbling of the mortar around a honeycombed area in the bottom. Slight scale appeared on the finished surface after 59 alternations.
		7	Pronounced scale on finished surface noticed after 30 alternations.
Vibrolithic process	6	2	The finished surface disintegrated to a depth of 1/8 to 1/4 inch. One corner disintegrated where it was honeycombed. First noted after 80 alternations.
		7	The finished surface disintegrated to a depth of 1/4 inch after 30 alternations. Some crumbling of the mortar noted after 80 alternations.
Do	17	2	Slight scale apparent on the finished surface after 80 alternations.
		7	Softening of the mortar noted at 2 small spots on the sawed surfaces after 30 alternations. The finished surface above the trap rock disintegrated to a depth of 1/8 to 1/4 inch after 80 alternations.
Electric vibrator	9	2	The finished surface scaled to a depth of 1/4 inch after 94 alternations.
		7	Slight scale noted on 1 sawed face after 30 alternations.
Do	25	2	No apparent effect. (One corner broken in handling.)
		7	Slight softening of 1 sawed face noticeable after 80 alternations.

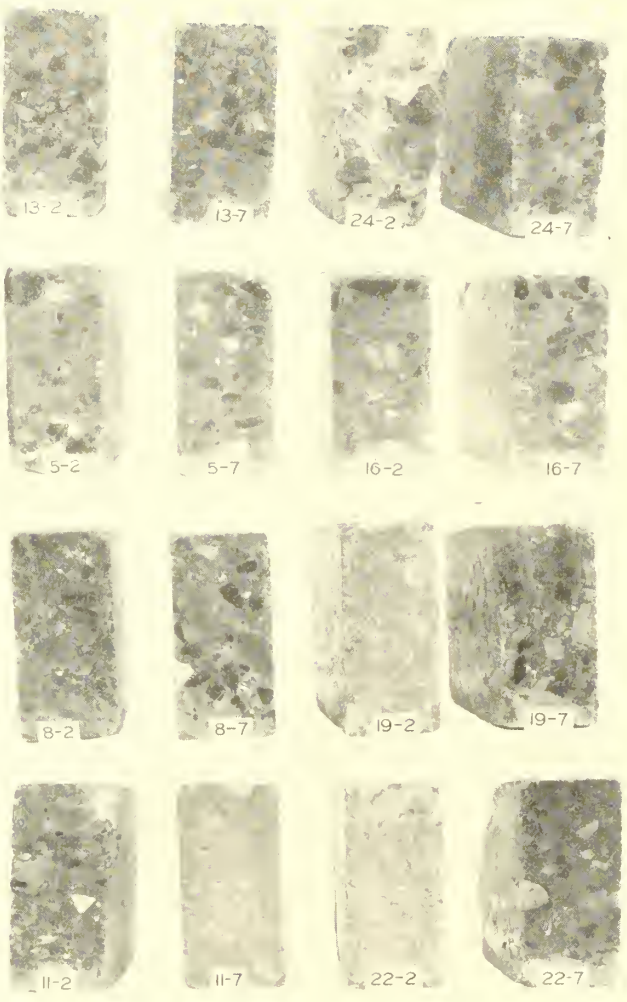


FIGURE 27.—RESISTANCE OF STONE CONCRETE SPECIMENS TO REPEATED FREEZING. SLAB AND SECTION NUMBERS ARE GIVEN ON THE PHOTOGRAPHS. SLABS 13 AND 24, HAND PLACED; 5 AND 16, VIBROLITHIC; 8 AND 19, ELECTRIC TAMPER; 11 AND 22, AIR HAMMER

particles of coarse aggregate which appeared to soften after repeated freezing. These particles were smooth, dark-colored, and noncellular in structure and had the appearance of having been incompletely fused during manufacture. The cellular particles were not affected by the repeated freezings. The mortar in the specimens containing the Haydite aggregates showed the effects of the test in practically all of the specimens. It is worthy of note, however, that the sawed surfaces which exposed the interior of the mass to the freezing action showed much less effect than did the finished surfaces.

It appears to be true of all of the specimens that the least resistant surface is that which was left by the finishing process on the upper surface of the slab.

The effect of the various methods of placing on the resistance to repeated freezing is limited, so far as these data indicate, to the effect that the respective methods have on the amount and condition of the mortar surface on the finished slab. The methods of placing which make use of vibration tend to bring fine material and water to the surface of the concrete. This was mentioned in the discussion of wear resistance as

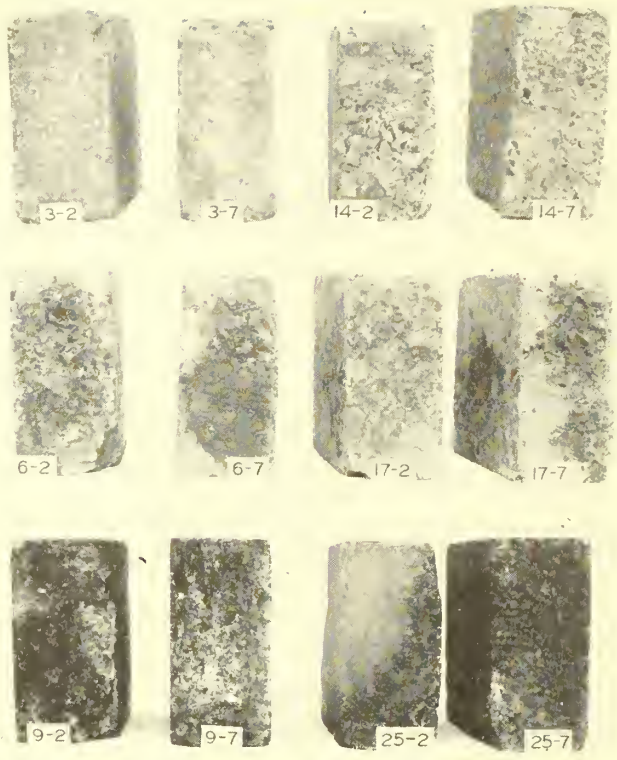


FIGURE 28.—RESISTANCE OF HAYDITE CONCRETE TO REPEATED FREEZING. SLAB AND SECTION NUMBERS ARE GIVEN ON THE PHOTOGRAPHS. SLABS 3 AND 14, HAND PLACED; 6 AND 17, VIBROLITHIC; 9 AND 25, ELECTRIC TAMPER

affecting the surface hardness of the concrete, and it is interesting to note that the surfaces left by the vibratory methods of placing were apparently less resistant to freezing than those obtained by the hand-placing method.

BULLETIN ISSUED ON ELECTRICAL EQUIPMENT FOR MOVABLE BRIDGES

A bulletin intended to assist bridge engineers in designing electrical equipment on movable bridges has recently been issued by the Bureau of Public Roads. The bulletin is designated as Technical Bulletin No. 265, Electrical Equipment on Movable Bridges, by Condé B. McCullough, Albin L. Gemeny, and W. R. Wickerham.

The first section of the bulletin deals with the fundamentals of direct-current and alternating-current motors and discusses the selection of motors for different services. The second section discusses the control and interlocking of operations, describing current practice in the regulation of motors, control of sequence of operations, and the use of various protective devices. This discussion is followed by a description of four different installations of electrical equipment, accompanied by diagrams showing the wiring and arrangement. The last portion of the bulletin deals with new developments in electrical bridge control, including variable-voltage control, speed-matching indicators for vertical-lift bridges, and light-sensitive relays.

Copies of this publication may be obtained from the Office of Information, United States Department of Agriculture, Washington, D. C.

HIGHWAY RESEARCH BOARD HOLDS ANNUAL MEETING

THE ELEVENTH annual meeting of the Highway Research Board was held on December 10 and 11 at the building of the National Academy of Sciences and the National Research Council, Washington, D. C. The session was opened at 10 a. m., Thursday, December 10, with an address of welcome by Dr. Albert L. Barrows, assistant secretary of the National Research Council, followed by an address by the chairman of the Highway Research Board, H. S. Mattimore, engineer of materials, Pennsylvania Highway Department. Mr. Mattimore presided at the meeting.

The Thursday morning session was devoted to the report of the committee on maintenance, B. C. Tiney, maintenance engineer, Michigan Highway Department, chairman; the report of a special investigation of dust-laying materials and methods, by Fred Burggraf, research engineer, Highway Research Board; and the report of the project committee on correlation of research in mineral aggregates, W. J. Emmons, director, Michigan State Highway Laboratory, chairman.

The afternoon meeting (C. M. Upham, of the American Road Builders' Association, presiding) was occupied with the following subjects: The report of the committee on materials and construction, H. S. Mattimore, chairman; the report of the project committee on curing of concrete pavement slabs, F. C. Lang, engineer of tests and inspection, Minnesota Highway Department, chairman; Compaction of Fills as Affected by Type and Size of Hauling and Other Equipment, by A. K. Haxtun, assistant engineer, American Road Builders' Association; Functions of Steel Reinforcement in Concrete Pavements and Bases, by C. A. Hogentogler, F. A. Robeson, and E. A. Willis, division of tests, United States Bureau of Public Roads; and the report of the special investigation of methods for evaluating field inspection of culverts, by R. W. Crum, director, Highway Research Board.

On Friday morning a short general session was held at which Mr. Mattimore presided. At this meeting the report of the committee on highway finance was delivered by Thomas H. MacDonald, chief, United States Bureau of Public Roads, chairman. The remainder of the day was occupied with two simultaneous sessions.

The session on highway design and allied subjects was held in the lecture room, E. F. Kelley, chief, division of tests, United States Bureau of Public Roads, presiding. This meeting was devoted to the report of the committee on highway design, A. T. Goldbeck, director, bureau of engineering, National Crushed Stone Association, chairman; Some Principles of Soil Surveying and Mapping, by D. P. Krynine, research associate in soil mechanics, Yale University; and the report of the special investigation of the use of rail steel reinforcement on highway construction, R. L. Morrison, chairman.

The session on traffic and transportation took place in the auditorium, with E. W. James, chief, division of highway transport, United States Bureau of Public Roads, in the chair. The following reports were considered: The report of the committee on traffic, G. E. Hamlin, superintendent of maintenance, Connecticut Highway Department, chairman; Field Methods for Measuring Tire Wear, by A. A. Anderson and H. B. Wright, Portland Cement Association; and the report of the committee on transportation, T. R. Agg, assistant dean of engineering, Iowa State College, chairman.

At 1 o'clock on Thursday, December 10, a luncheon for contact men was held in the library of the National Academy of Sciences. The annual highway research dinner was held at 7 in the evening of the same day, at the Willard Hotel.

Space does not permit an adequate account of the many excellent papers submitted to the Highway Research Board at this meeting. The following paragraphs contain brief summaries of a number of reports which are considered of particular interest to highway engineers.

MAINTENANCE

Maintenance Costs, by H. L. Bishop, chief, division of construction, United States Bureau of Public Roads, discussed progress on a study of maintenance costs on Federal-aid projects in all the States. Because of lack of uniformity in accounting methods, the Bureau of Public Roads has adopted a system of rating the maintenance on Federal-aid projects by weighing the factors of surface, drainage, shoulders, structures, roadside cleaning, and traffic service according to their relative importance. Other articles on maintenance included Maintenance of Expansion Joints and Cracks in Concrete Pavements, by W. H. Root, maintenance engineer, Iowa Highway Commission; Fillers and Bedding Courses for Brick Pavements, by J. S. Crandall, professor of highway engineering, University of Illinois; and Ice Removal from Pavements, by B. C. Tiney, maintenance engineer, Michigan Highway Department.

MATERIALS AND CONSTRUCTION

H. S. Mattimore, engineer of materials, Pennsylvania Highway Department, reporting for the committee on materials and construction, submitted a paper on The Effect of Hot Cement on Time of Set and Concrete Quality, in which he discussed the effects produced by the use of cement which when received on the work retains a considerable part of the heat generated during manufacture. Occasional temperatures as high as 150° to 200° F. have been noted. In The Durability of Concrete as Affected by the Water-Cement Ratio, Mr. F. H. Jackson analyzed the evidence concerning the relation between resistance to frost action and the water-cement ratio, the maximum water-cement ratio allowable for concrete exposed to the weather, and the relation between strength and durability. A decided relation was found to exist between the water-cement ratio and durability. There was also a committee report on Volume Changes in Concrete, by C. H. Scholer, professor of applied mechanics, Kansas State College.

The paper on Functions of Steel Reinforcement in Concrete Pavements, by Messrs. Hogentogler, Robeson, and Willis pointed out the fact that investigations of steel reinforcement carried on by the Highway Research Board in 1924 and 1925 disclosed benefits furnished by reinforcement on concrete pavements not readily explainable by the conventional theory of reinforced concrete design. Subsequent study indicates that some of the benefits are due to the influence the rein-

forcement exerts upon the internal stresses and other factors which affect the strength and durability of concrete. The report analyzed the phenomena occurring during and subsequent to the hardening period and discusses the successive shrinkage, expansion, warping, and other distortions which may occur as the result of natural causes as well as traffic. The attempt was made to explain the function of steel reinforcement with respect to the stresses caused by these phenomena.

FINANCE

Mr. MacDonald's paper on highway finance dealt with the cooperative project being conducted by the Bureau of Public Roads and the University of Wisconsin on The Relationship Between Road and General Taxes in the Various States. The first work of the investigation was carried on in Wisconsin and was summarized in the report.

HIGHWAY DESIGN

The report of the committee on highway design included a symposium on The Economics of Low Cost Bridges: Concrete, by E. M. Fleming, Portland Cement Association; Timber, by J. F. Seiler, American Wood Preservers' Association; and Steel, by F. H. Frankland, American Institute of Steel Construction. Mr. Fleming's paper analyzed methods of reducing first cost by the use of simpler types of concrete structures, higher strength concrete, rigid frame structures, and continuous slab viaducts. After analyzing methods of calculating the total carrying cost of wooden bridges, Mr. Seiler cited experience to show that creosoted timber will last from 35 to 40 years, after which time a bridge would probably be retired from obsolescence. Mr. Frankland gave examples of recent economical designs of steel bridges using continuous girders and beams and steel-plate floors, as well as the use of steel piles for substructures. The life of steel structures was placed at not less than 100 years.

Load Limitations on Highways, by Albin L. Gemeny, senior structural engineer, United States Bureau of Public Roads, discussed the relation between load limitations on roads and those on bridges, and presented a discussion of load limitations based on the design loads of the American Association of State Highway Officials.

Field Experiments in Subgrade Drainage and Treatment, by F. H. Eno, research professor of highway engineering, Ohio State University, discussed an investigation of 42,500 feet of porous or treated subbases beneath paved roads and approximately 25,700 feet of untreated paved roads, the comparisons being made on the basis of crack ratios. Upon pavements with no center joint, slag subbases yielded no longitudinal cracks; sand gave a longitudinal crack ratio of 0.095; gravel, 0.2; a cement-clay mixture, 0.29, untreated sections, 0.6.

In A New Theory of Frost Heaving, A. C. Benkelman, research engineer, and F. R. Olmstead, research assistant, Michigan Highway Department, analyzed the factors which cause the formation of ice plates in soil, and noted the elimination of frost heaving at some 200 locations in Michigan last winter through provision for drainage.

The article by D. P. Krynine, on Principles of Soil Surveying and Mapping for Road Purposes, recommended that in addition to the soil grouping defined by the Bureau of Public Roads the soils be further characterized with respect to permeability and stability. For the former the author suggested a method for determining the moisture equivalent of the soil by centrifuging, and for the latter a new test for shearing resistance.

TRAFFIC AND TRANSPORTATION

Papers on the general subject of traffic included Traffic Capacity, by A. N. Johnson, dean of engineering, University of Maryland, a discussion of a cooperative project by the Bureau of Public Roads and the University of Maryland to determine the capacity of 2, 3, and 4 lane roads; Vehicle and Highway Mechanics, by Dr. H. C. Dickinson, chief, heat and power division, United States Bureau of Standards; Traffic Survey Procedure, by W. Graham Cole, policyholders service bureau, Metropolitan Life Insurance Co.; Law Observance and Enforcement, by Burton W. Marsh, traffic engineer, Philadelphia; Highway Education, by Stephen James, director of extension, Highway Education Board; State Control of Traffic, by W. A. Van Duzer, director of traffic, District of Columbia; and The Driver, by Sidney J. Williams, director, public safety division, National Safety Council.

The committee on transportation submitted three papers: Tractive Resistance Studies with a Gas-Electric Automobile, by Ray Paustian, Iowa State College; Tractive Resistance and Tractive Effort of Motor Vehicles, by H. B. Shaw, director, engineering experiment station, North Carolina State College; and Air Resistance of Motor Vehicles, by W. E. Lay, professor of mechanical engineering, University of Michigan. Mr. Paustian's paper discussed the equipment and preliminary results obtained in measurements of tractive resistance, wind resistance, and power consumption at speeds up to 65 miles per hour. Mr. Shaw gave methods for computing tractive resistance and tractive effort at different speeds and grades. Professor Lay reported on wind-tunnel experiments with simple body forms, using different methods of simulating a road surface. The data showing the percentage of improvement due to stream-lining proved to be fairly consistent for all methods tried.

SUBGRADE REPORTS REPRINTED IN BULLETIN FORM

The four reports on the subject of subgrades which were published in the June, July, September, and October issues of PUBLIC ROADS have been reprinted under the title, Reports on Subgrade Studies. The following articles are included: Subgrade Soil Constants, Their Significance and Their Application in Practice (Parts I, II, and III); The Soil Profile and the Subgrade Survey; Procedures for Testing Soils for the Determination of the Subgrade Soil Constants; Graphical Solution of the Data Furnished by the Hydrometer Method of Analysis.

Single copies of this publication may be obtained from the United States Bureau of Public Roads, Willard Building, Washington, D. C. In case more than one copy is desired, they may be purchased from the Superintendent of Documents, United States Government Printing Office, Washington, D. C., at 40 cents each.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.

Report of the Chief of the Bureau of Public Roads, 1925.

Report of the Chief of the Bureau of Public Roads, 1927.

Report of the Chief of the Bureau of Public Roads, 1928.

Report of the Chief of the Bureau of Public Roads, 1929.

Report of the Chief of the Bureau of Public Roads, 1931.

DEPARTMENT BULLETINS

No. *136D. Highway Bonds. 20c.

*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.

*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.

*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c

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1486D. Highway Bridge Location.

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TECHNICAL BULLETINS

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SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y. Road Work on Farm Outlets Needs Skill and Right Equipment.

MISCELLANEOUS CIRCULARS

No. 62MC. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-Aid Highway Projects.

*93MC. Direct Production Costs of Broken Stone. 25c.

109MC. Federal Legislation and Regulations Relating to the Improvement of Federal-Aid Roads and National-Forest Roads and Trails, Flood Relief, and Miscellaneous Matters.

MISCELLANEOUS PUBLICATION

No. 76MP. The Results of Physical Tests of Road-Building Rock.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio. (1927)

Report of a Survey of Transportation on the State Highways of Vermont. (1927)

Report of a Survey of Transportation on the State Highways of New Hampshire. (1927)

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio. (1928)

Report of a Survey of Transportation on the State Highways of Pennsylvania. (1928)

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No 17, D– 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.

Vol. 5, No. 19, D– 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.

Vol. 5, No. 24, D– 6. A New Penetration Needle for Use in Testing Bituminous Materials.

Vol. 11, No. 10, D–15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

NOVEMBER 30, 1931

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage	MILEAGE Total	Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total		
		$	$				$	$				$	
Alabama	2,387.8	1,777,150.13	864,457.19	80.7		80.7						5,345,824.09	Alabama
Arizona	1,076.9	4,078,388.78	2,910,116.84	192.0	64.7	256.7	215,102.75	161,950.85	20.0	19.8	39.8	1,743,928.64	Arizona
Arkansas	1,889.3	4,328,556.75	2,130,261.65	104.8	85.6	190.4	362,032.53	181,016.26	17.3		17.3	1,967,317.90	Arkansas
California	2,142.0	11,715,639.73	4,827,535.30	290.7	36.7	327.4	1,468,368.92	636,146.13	36.8	3.6	40.4	3,408,511.66	California
Colorado	1,452.9	6,241,669.16	3,177,185.41	268.6	76.7	345.2	722,164.01	323,854.09	8.6	4.6	13.1	2,078,778.86	Colorado
Connecticut	282.7	3,627,221.97	1,279,199.52	28.7		28.7						706,266.13	Connecticut
Delaware	361.3	206,946.00	103,473.00	4.6		4.6	108,131.00	54,065.50	9.7		9.7	516,622.43	Delaware
Florida	554.4	6,147,898.61	2,872,456.15	174.4		174.4						3,046,471.31	Florida
Georgia	2,978.2	7,001,386.58	3,171,723.08	168.5	157.8	326.3	884,530.69	439,999.26	34.8	21.0	65.8	2,366,149.47	Georgia
Idaho	1,431.9	2,379,675.26	1,396,198.93	130.9	90.2	221.1	98,447.53	58,388.61	5.0		5.0	1,479,979.03	Idaho
Illinois	2,641.7	22,745,509.64	10,488,709.14	680.5	28.7	709.2	4,656,663.95	2,093,162.65	145.4	4.7	150.1	3,700,407.94	Illinois
Indiana	1,726.3	8,738,423.39	4,331,011.54	261.1		251.1	1,706,186.78	812,731.06	61.6	12.6	74.1	2,648,193.74	Indiana
Iowa	3,348.6	1,220,570.06	390,966.47	20.5	22.3	42.8	274,243.43	119,960.00	5.6	5.4	11.0	2,768,864.32	Iowa
Kansas	3,630.4	3,582,250.77	1,660,993.87	172.8	23.1	195.9	39,928.88	18,029.02	3.3		3.3	3,076,169.68	Kansas
Kentucky	1,784.8	3,490,600.04	1,595,570.07	201.0	27.4	228.4	70,254.64	36,127.31	.6	4.0	4.5	2,029,039.79	Kentucky
Louisiana	1,484.0	7,114,787.76	3,317,899.49	168.6	11.7	180.3	2,667,840.66	135,581.66	1.0		1.0	1,644,886.88	Louisiana
Maine	883.6	3,061,521.21	1,317,172.11	63.9		63.9	37,646.34	14,086.00	.9		.9	1,285,660.42	Maine
Maryland	762.2	806,057.96	331,912.28	21.3		21.3						911,088.46	Maryland
Massachusetts	754.8	12,589,382.69	3,727,732.44	100.4		100.4	1,463,156.49	494,520.66	17.6		17.6	1,080,261.61	Massachusetts
Michigan	1,926.2	10,176,759.43	4,439,342.63	353.8	49.8	403.6	779,121.45	363,477.50	32.0	2.5	34.5	3,921,044.25	Michigan
Minnesota	4,208.3	2,777,438.57	1,163,826.98	28.4	118.4	146.8	3,378,002.69	1,139,480.17	87.4	67.8	155.2	1,800,857.50	Minnesota
Mississippi	1,808.1	3,540,448.12	1,749,977.28	167.8	61.3	229.1	415,088.00	184,234.59		17.3	17.3	5,515,254.55	Mississippi
Missouri	2,869.6	4,881,374.86	2,009,012.56	130.4	20.8	151.2	1,225,128.41	544,039.45	46.7		46.7	3,062,740.27	Missouri
Montana	2,494.0	6,523,994.08	3,655,972.26	480.9	40.4	521.3	622,013.88	348,436.03	70.0	12.4	82.4	3,180,014.93	Montana
Nebraska	4,097.5	7,145,232.06	3,363,577.67	248.1	94.9	343.0	25,454.82	12,727.40		18.4	18.4	2,592,325.16	Nebraska
Nevada	1,208.1	2,149,725.38	1,456,085.40	64.8	193.1	257.9						1,667,343.81	Nevada
New Hampshire	407.4	1,101,793.56	407,090.46	21.7	4.3	26.0						534,433.03	New Hampshire
New Jersey	573.7	6,777,396.56	2,435,375.14	67.6	.8	68.4						1,699,922.94	New Jersey
New Mexico	2,245.9	1,625,133.61	1,158,777.51	67.4	11.7	79.1	119,577.99	75,641.82	2.4		2.4	1,766,406.18	New, Mexico
New York	2,996.5	26,026,020.00	9,958,297.60	615.0		515.0	1,590,000.00	639,750.00	35.8		35.8	5,307,148.59	New York
North Carolina	2,180.6	2,276,820.34	1,113,982.67	91.3	7.4	98.7	261,394.20	121,927.92	12.4	8.0	20.4	3,865,293.35	North Carolina
North Dakota	5,074.3	1,884,653.78	945,968.83	151.6	226.1	377.6	1,015,133.41	623,049.78	133.7	130.2	263.9	2,107,901.63	North Dakota
Ohio	2,728.8	10,627,774.18	3,786,966.77	182.8	38.3	221.1	882,244.81	364,645.30	13.8	.4	14.2	4,395,480.71	Ohio
Oklahoma	2,139.4	6,013,724.70	2,940,380.04	170.2	106.1	276.3	40,576.56	22,438.83	.1		.1	2,533,407.28	Oklahoma
Oregon	1,614.1	3,566,920.57	1,991,150.18	115.1	35.6	150.7						2,200,463.19	Oregon
Pennsylvania	2,931.4	6,364,129.44	2,942,184.66	126.4		126.4	15,163.78	7,040.14	.1		.1	5,151,715.52	Pennsylvania
Rhode Island	249.7	1,087,677.93	309,140.62	14.3		14.3						662,607.52	Rhode Island
South Carolina	1,959.4	3,531,650.70	1,560,984.08	76.8	67.9	144.7						1,389,774.57	South Carolina
South Dakota	3,961.7	3,509,204.53	2,041,080.79	234.8	189.6	424.3	11,839.81	6,674.64				1,613,069.03	South Dakota
Tennessee	1,666.6	526,796.35	263,398.15	10.2	4.3	14.5	892,398.31	429,604.69	30.3	10.7	41.0	3,799,566.22	Tennessee
Texas	7,480.2	14,305,362.68	6,669,606.28	694.4	188.9	883.3	2,039,466.25	884,609.59	95.1	43.4	138.5	7,806,467.39	Texas
Utah	1,194.4	1,109,093.76	682,133.30	68.8	13.9	82.7	50,994.68	37,736.05	5.7		5.7	1,526,514.60	Utah
Vermont	321.2	787,303.55	341,347.86	22.3		22.3						541,920.57	Vermont
Virginia	1,796.2	3,981,506.27	1,846,614.77	180.3	22.0	202.3	619,850.20	309,768.55	21.9	7.3	29.2	2,031,764.06	Virginia
Washington	1,160.7	2,562,758.42	1,219,155.33	81.7	6.5	88.2	425,552.72	228,800.00	10.5		10.5	1,911,634.96	Washington
West Virginia	855.7	2,713,732.03	1,214,862.30	72.9	10.5	83.4	97,329.39	40,854.05	.6		.6	1,264,883.70	West Virginia
Wisconsin	2,681.0	5,909,497.23	2,171,619.63	179.4	60.3	239.7	15,741.75	6,000.00	.4		.4	2,699,243.18	Wisconsin
Wyoming	2,030.5	2,098,323.95	1,220,908.47	163.4	71.6	235.0	142,062.76	75,614.83	17.5	11.8	29.3	1,296,072.81	Wyoming
Hawaii	59.4	1,094,751.98	481,061.26	31.6		31.6	82,324.99	22,869.00	1.6		1.5	2,044,877.80	Hawaii
TOTALS	97,993.4	257,528,574.98	115,428,404.61	7,928.0	2,268.3	10,196.3	29,422,047.25	11,965,598.17	986.0	405.7	1,391.7	121,494,541.35	TOTALS

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 12, NO. 11 ▽ JANUARY, 1932

PAVING OPERATIONS ON MOUNT VERNON MEMORIAL HIGHWAY

For sale by the Superintendent of Documents, Washington, D. C. - - - - - - - - - - See page 2 of cover for prices

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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The reports of researeh published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

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EFFECT OF SIZE OF BATCH AND LENGTH OF MIXING PERIOD ON RATE OF PRODUCTION AND QUALITY OF CONCRETE MIXED IN STANDARD 27E PAVERS

Reported by T. C. THEE, Assistant Highway Engineer, Division of Management, U. S. Bureau of Public Roads

THE rate at which any given concrete paver can produce concrete depends very largely on the size of the batch and the length of time which it must be mixed. Definite data as to the relation which the size of batch, number of sizes of coarse aggregate, and the length of mixing bears to the rate of production and to the quality of the concrete produced, when larger than normal batches are used, have been lacking. A rather extensive study was undertaken in 1930 to determine, if possible, how large a batch can safely be handled by standard 27E pavers, under present operating conditions and with present mixing time specifications, without any detriment to the strength and uniformity of the mix. It was also desired to learn whether or not concrete which is of satisfactory strength and uniformity of mix can be produced with 27E pavers using a 33-cubic-foot batch and a mixing time of 60 seconds or less.

The projects selected for this study involved rather extreme conditions and are described in the following paragraphs.

Two Wisconsin projects were studied, State-aid project 2916, 9.88 miles in length, and State-aid project 2926, 11.27 miles, both in Sheboygan County. The following description applies to both jobs: 20-foot section, 9–6.5–9 inch thickness; a normal concrete having a slump of about 1½ inches, a constant cement and water content, and a practically constant proportion of gravel and sand, three sizes of coarse aggregate uniformly graded with a maximum size of 2½ inches and a workability factor, b/b_0, averaging 0.75 and varying by design hardly an appreciable amount. The aggregates were proportioned by weight and bulk cement was used. The water devices on the four different mixers used on these two jobs were fairly accurate. The blades and buckets in all the mixers were new and in excellent condition.

A study was also made on Federal-aid project 259 A and B, Jefferson County, Ark. Following is a description of the project: Length, 16.97 miles; 18-foot section, 9–6–9 inch thickness; a relatively dry concrete with an approximate slump of 1¼ inches, a constant cement and water content, and a fairly uniform sand and stone content, two sizes of well-graded, crushed trap rock as coarse aggregate, with a maximum size of 2½ inches, and a workability factor, b/b_0, averaging approximately 0.75 and varying only within a narrow range by design. The aggregates were proportioned by weight and sack cement was used. A dual water tank open to atmospheric pressure measured the water fairly accurately. The blades and buckets of the mixer drum were in fair condition.

It is believed that these three jobs are fairly representative of present good practice in the production of concrete for highway paving purposes in the United States, and that the most probable dangers or difficulties that are likely to arise in connection with the use of batches larger than those normally used, a reduced mixing time, and multiple-sized aggregates, would be evidenced on one or more of these jobs during the course of the studies.

SIZE OF BATCH VARIED

On the Wisconsin jobs the contractors presented alternate bids for constructing the pavement when using 27, 30, 33, and 35 cubic-foot batches to an extent sufficient to provide for the construction of at least 1 mile of concrete with each of these different sized batches when using a mixing time of 60 seconds. Although no provisions had been made in the bids to use other than a 60-second mixing time, near the close of the jobs the batchmeter was actually set at 50, 60, and 80 seconds for both 30 and 33 cubic-foot batches.

On State-aid project 2926 the successful contractor bid the following:

Size of batch	Bid per square yard [1]
27 cubic feet	$0.97
30 cubic feet	.94
33 cubic feet	.93
35 cubic feet	.93

On State-aid project 2916 the successful contractor bid the following:

Size of batch	Bid per square yard [1]
27 cubic feet	$1.04
30 cubic feet	.99
33 cubic feet	.97
35 cubic feet	.98

Prior to this time the maximum allowable batch in Wisconsin was 30 cubic feet, and the contractors did not reflect as large a reduction in bid prices between the 30 and 33-cubic-foot batches as between the 27 and 30 cubic foot batches, probably because there was some doubt in their minds concerning the ability to handle the larger batches in the trucks or in the mixer. As the large sized batches were tried out in actual operation, the contractors on these jobs found that the 33-cubic-foot batch was handled with as much ease as the smaller batches, and even the 35-cubic-foot batches were handled by the new mixers without any difficulty.

On the Arkansas job the investigation was primarily arranged for quality and not for production, although detailed stop-watch studies were made on the different batch sizes to determine how the mixer cycle was affected.

UNIFORM PROCEDURE ADOPTED FOR OBTAINING TEST SAMPLES

In order to obtain data on these three jobs as nearly comparable as possible, the same general procedure of sampling and testing was used on all jobs in determining, for each of the four corners and the center of each batch, the exact amount of gravel, sand, cement, and water contained in the concrete at these respective points. One cylinder for a compression test was also made from the concrete taken at each of these points. Three beams were made from each sampled batch and the molds for these were so placed that, in general, a beam break would be obtained for each of the five points from which the cylinders were taken.

Figure 1 shows the five sample buckets and three beam molds placed on the subgrade. The spreader bucket was dumped over these buckets and beam molds in the same way on all the jobs, so that the first beam mold

[1] State furnished cement.

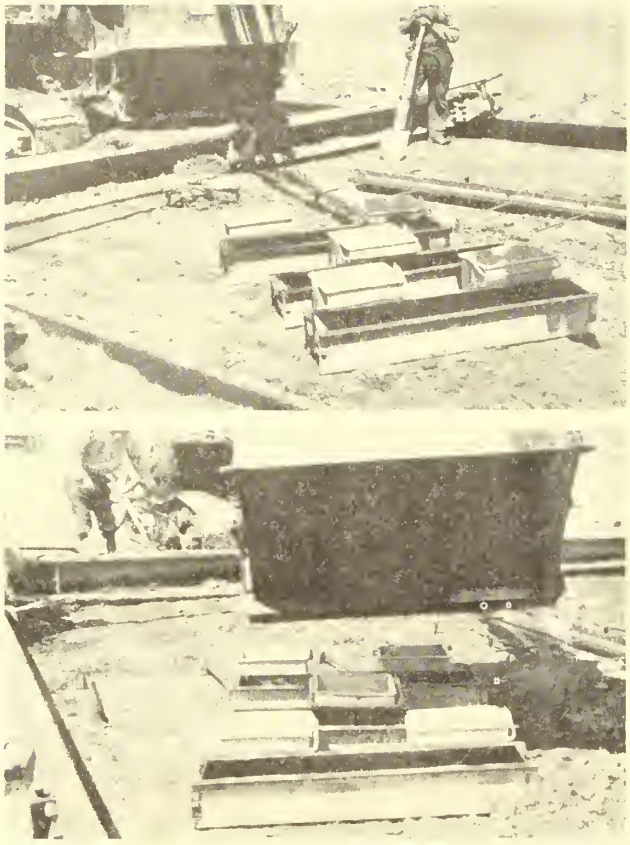


FIGURE 1.—METAL SAMPLE BUCKETS AND METAL BEAM MOLDS IN POSITION TO RECEIVE CONCRETE AS DUMPED FROM MIXER SPREADER-BUCKET

and two buckets received the first part of the batch, the next beam mold and bucket was filled from the middle part of the batch and the remaining beam mold and buckets received samples from the last part of the batch. As soon as this operation was completed, the sample buckets and beam molds, properly identified, were removed to the berm of the roadway as shown in the upper left-hand photograph of Figure 2. The procedure was then as follows: The concrete obtained in the first bucket was dumped into a large pan and small scoopfuls of concrete were placed alternately in a cylinder and in a pail for the separation test. This operation was repeated consecutively for each of the other buckets. The cylinder molds rested on a steel plate and were arranged in the same order as that in which the buckets were placed on the subgrade. Each cylinder was tamped twenty times along the edge and five times at various places in the center of each one-third point as they were being filled.

While these operations were in progress another man spaded the beams twenty times along each side and four times along each end with a trowel, then rodded along the edges with a bullet-pointed ⅝-inch rod in a like manner, and finally repeated the spading with the trowel after which the surface was struck off and finished. Every effort was made to leave the mass of concrete in the center of the beams undisturbed. These specimens, both beams and cylinders, were cured on the berm under wet burlap about 24 hours and then hauled in damp sand to the central curing point. In Arkansas the specimens were cured near the job in a large lake which had practically a constant temperature of about 80° F. In Wisconsin the specimens were cured in wet sand which had a fairly constant temperature of about 80° F. The beams on all the jobs were cured at the field curing station until broken. The cylinders were left to cure for about 21 days and then hauled to the State testing laboratory, where they were placed in a moist closet until they were broken at 28 days. All the beams on these four jobs were broken by the same type of portable cantilever testing machine and on each job the same operator made all the breaks. In testing the beams the load was applied on the dynamometer at the rate of 40 pounds in 10 seconds. (See fig. 2.)

PROPORTIONS OF MATERIALS IN EACH SAMPLE DETERMINED BY WASH TEST

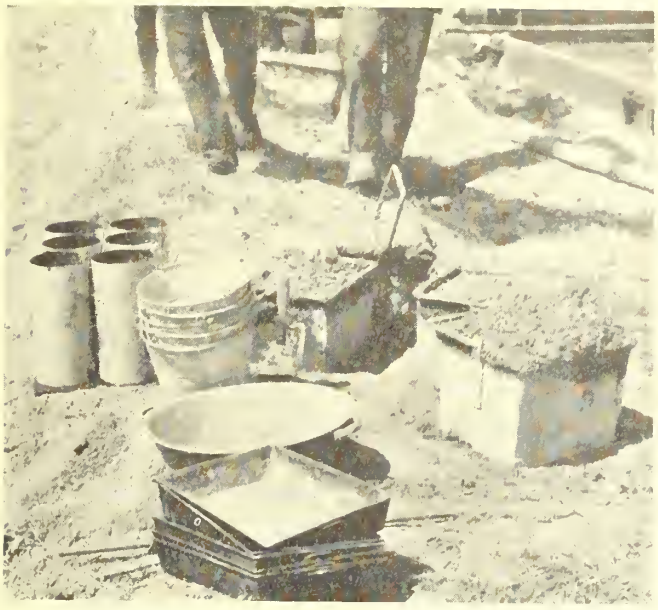
A 25-pound sample of concrete was always used for the separation or wash test. All weighing was done on a 35-pound scale sensitive to one-sixteenth of an ounce. All weights were recorded in ounces and fractions of an ounce. The procedure was as follows: Each sample representing one of the four corners of the batch or the center was immediately weighed and adjusted to 25 pounds in air, and then weighed under water. (See fig. 3.) It was then placed in a nest of sieves, consisting of one No. 4, one 48-mesh, and one 100-mesh sieve, and washed over a large tub in order to retain all the wash water. The material retained on the No. 4 sieve was classified as coarse aggregate, and that retained on the 100-mesh sieve as sand. The weight of sand was later corrected to include the weight of material passing the 100-mesh sieve, which was determined by a separate auxiliary test. This material was first weighed under water, then air dried, weighed, and subjected to a sieve analysis. The sand was also weighed directly under water. The weight of the cement under water was then computed by obtaining the difference between the weight of the total sample under water and the sum of the corrected weights of the gravel and sand under water. The weight of cement for the center sample was checked by permitting the cement washed from the center sample to settle in the tub and weighing the cement thus collected under water. The weight of water was obtained by taking the difference between the total sample, or 400 ounces, and the sum of the weights of the gravel, sand, and cement. (See fig. 4.) The specific gravity of the gravel and the sand was determined with a metal pycnometer.

In order to determine the percentage of moisture in the aggregates, samples of the gravel and sand for each batch tested were taken at the plant while the truck was loading, and tested. This sample was then washed through a nest of sieves to determine the correction factor for the material passing the 100-mesh sieve.

DETAILED PRODUCTION STUDIES MADE ON WISCONSIN JOBS

On the two Wisconsin jobs daily detailed production studies were made. At least a mile of pavement was constructed for each batch size of 27, 30, 33, and 35 cubic feet. The batchmeter was generally set at 60 seconds, which resulted in an actual mixing time of about 55 seconds for all solid materials. A little over a mile of pavement was also constructed on which a 33-cubic-foot batch was used, with the batchmeter set at 50 seconds, making the actual mixing time of all solid materials about 45 seconds.

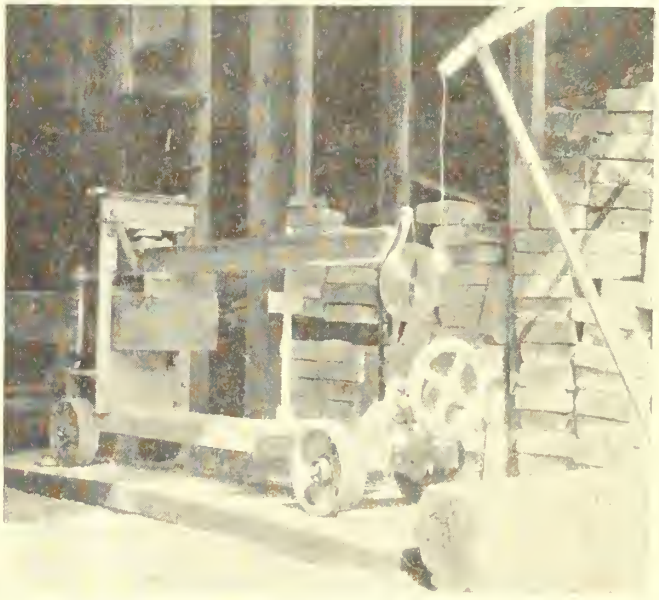
By means of stop-watch studies of the key equipment for two or more hours every day, the effect on production and unit costs was determined for each batch size and each mixing time. Three to four batches were usually sampled each day. The different sized batches



REMOVING SAMPLE BUCKETS AND BEAMS FROM THE SUBGRADE TO THE BERM OF THE ROAD, WHERE ANALYSIS WAS MADE OF THE FIVE PARTS OF THE BATCH FOR THE EXACT AMOUNT OF GRAVEL, SAND, CEMENT, AND WATER.

CYLINDERS WERE MADE ACCORDING TO THE A. S. T. M. METHODS. BEAMS WERE RODDED ONLY AT THE EDGES WITHOUT DISTURBING THE CENTER MASS OF THE CONCRETE.

BEAM SPECIMENS WERE CURED IN DAMP SAND FOR 28 DAYS AND TESTED IN THE FIELD.



THE BEAM-TESTING MACHINE AND THE LOAD APPLICATION WERE CONTROLLED SO AS TO BE THE SAME ON ALL SPECIMENS, AND ALL TESTS WERE MADE BY THE SAME OPERATOR.

FIGURE 2.—SAMPLES FOR TEST CYLINDERS AND WASH ANALYSES WERE OBTAINED FROM 4 CORNERS AND CENTER OF EACH BATCH AS PLACED; THREE BEAMS REPRESENTATIVE OF THE SAME PORTIONS OF THE BATCH WERE ALSO CAST

were alternated in order to make all conditions comparable. For example, in the morning a 27-cubic-foot batch would be sampled; just before noon a 30-cubic-foot batch; immediately after noon a 33-cubic-foot batch; and later a 35-cubic-foot batch. The following day this order would be changed. When batches to be sampled were either smaller or larger than those being run, about nine to twelve batches of the new size would be run through the mixer before the sample was taken so that the mixer would be operating normally under the changed batch size. The water at the mixer was always changed in proportion to the batch size being tested.

As soon as the work in Wisconsin was well under way the quality tests and production studies indicated that it would be decidedly more economical to use a 33 than a 30-cubic-foot batch, and that this could be done without any sacrifice of quality in regard to either uniformity of mix or strength. The 33-cubic-foot batch was therefore adopted and used entirely, except for such few modifications as were necessary in order to obtain the required test samples.

Three-year-old mixers, 1927 models, were used by both the contractors on the Wisconsin jobs during the first part of the studies. A complete series of tests and production studies were made on both of these jobs while the old pavers were still in operation, using 27, 30, 33, and 35 cubic-foot batches. Later the old mixers were replaced with new 1930 model 27E mixers and another complete study of 27, 30, 33, and 35 cubic-foot batches was made on each job for quality and production.

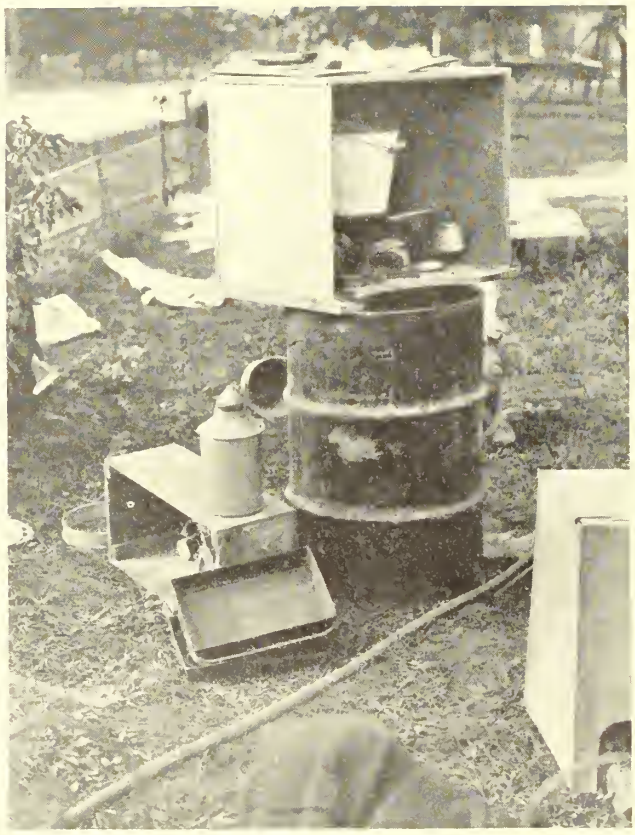


FIGURE 3.—EACH SAMPLE WAS WEIGHED IN AIR AND THEN IN WATER, WITH THE APPARATUS SHOWN. WEIGHING WAS ACCURATE TO ONE-SIXTEENTH OUNCE

FIGURE 4.—FIELD EQUIPMENT FOR WASH TEST ANALYSIS. LOWER PHOTOGRAPH SHOWS PROCESS OF WASHING THE CEMENT OUT OF THE SAMPLES OF CONCRETE THROUGH A NEST COMPOSED OF A NO. 4, A 48, AND A 100-MESH SIEVE

Before the Wisconsin studies were closed a series of tests with 30 and 33 cubic-foot batches was tried, using respectively, a 50, 60, and 80 second mixing time setting of the batchmeter. A complete study was made for each of these combinations. In all of this investigation the batchmeter was set at the nominal mixing time of 50, 60, or 80 seconds, so that the actual mixing time with all the solid materials in the drum was, in reality, about 45, 55, and 75 seconds, respectively.

DETAILED AND SUMMARY TABLES PREPARED

Detailed tables and summaries of which Table 1 is an example, were made for the studies on each job.

TABLE 1.—*Effect of size of batch on uniformity of mix and strength of concrete, for State-aid project 2926, Sheboygan County, Wis.; 27-cubic-foot batch; 27E paver, old model, good condition; aggregates, good limestone gravel, pit sand*

Batch No. and date made (1930)	Part of batch	Proportions by weight				Sample variation factor	Workability factor b/b_0	Cylinders, 28 days		Beams					
										7 days		28 days			
												End break		Center break	
		Gravel	Sand	Cement	Water			Strength	Variation	Strength	Variation	Strength	Variation	Strength	Variation
		Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*		*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*
No. 4 (Aug. 4)	A	49.34	32.75	11.02	6.88	1.68	0.691	2,950	4.90	605	4.01	782	2.21	805	2.98
	B	49.31	31.25	12.42	7.01	4.41	.691	2,840	8.44						
	Center	50.32	32.32	10.65	6.70	1.98	.708	3,330	7.35	540	7.17	857	7.16	815	4.20
	C	50.69	31.05	11.62	6.64	2.53	.713	3,200	3.16	600	3.15	760	4.96	725	7.25
	D	48.52	34.12	10.87	6.47	3.96	.684	3,190	2.84						
Average		49.64	32.30	11.32	6.74	2.91	.697	3,102	5.34	582	4.78	800	4.78	782	4.83
Batch variation	per cent	1.38	2.84	4.99	2.43										
No. 7 (Aug. 6)	A	52.53	30.70	11.31	5.46	2.16	.749	3,030	.03	638	16.01	835	5.70	780	4.29
	B	50.81	31.87	11.86	5.45	4.91	.724	3,230	6.64						
	Center	50.86	32.07	11.86	5.20	3.83	.727	2,695	11.02	525	4.54	735	6.97	790	3.07
	C	58.34	26.80	10.35	4.51	10.70	.845	2,900	4.26	487	11.46	800	1.27	875	7.37
	D							3,290	8.61						
Average		53.13	30.36	11.35	5.15	5.40	.761	3,029	6.11	550	10.67	790	1.65	815	4.91
Batch variation	per cent	4.89	5.85	4.53	6.33										
No. 10 (Aug. 17)	A	50.37	31.75	11.62	6.25	2.22	.713	3,080	1.48	510	1.30	800	4.19	770	5.67
	B	50.17	31.82	11.85	6.15	2.47	.750	3,160	4.12						
	Center	51.86	30.35	11.67	6.11	.80	.736	2,945	2.96	480	7.10	885	5.99	824	.94
	C	54.14	28.95	10.96	5.95	4.53	.767	2,810	7.41	560	8.39	820	1.80	855	4.76
	D	51.90	30.40	11.45	6.24	.84	.734	3,180	4.78						
Average		51.69	30.65	11.51	6.14	2.17	.740	3,035	4.15	517	5.60	835	4.00	816	3.79
Batch variation	per cent	2.19	2.95	2.12	1.43										

TABLE 1.—*Effect of size of batch on uniformity of mix and strength of concrete, for State-aid project 2926, Sheboygan County, Wis.; 27-cubic-foot batch; 27E paver, old model, good condition; aggregates, good limestone gravel, pit sand* Continued

Batch No. and date made (1930)	Part of batch	Proportions by weight				Sample variation factor	Workability factor b/b_0	Cylinders, 28 days		Beams					
										7 days		28 days			
												End break		Center break	
		Gravel	Sand	Cement	Water			Strength	Variation	Strength	Variation	Strength	Variation	Strength	Variation
		Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*		*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*
No. 14 (Aug. 8)	A	48.20	32.75	12.37	6.67	6.26	0.686	3,290	1.48	480	11.23	970	8.50	865	3.59
	B	52.51	30.30	11.47	5.71	3.61	.752	3,110	5.18						
	Center	50.48	30.72	12.15	6.64	2.92	.708	3,310	2.10	552	2.09	857	4.13	820	1.80
	C	53.81	29.12	11.17	5.83	5.11	.767	3,070	5.30	590	9.12	855	4.36	820	1.80
	D	51.34	30.45	12.30	5.91	2.09	.738	3,130	3.45						
Average		51.27	30.67	11.89	6.16	4.00	.730	3,242	3.50	541	7.48	894	5.66	835	2.40
Batch variation	per cent	3.00	2.78	3.84	6.37										
No. 23 (Aug. 13)	A	52.19	30.45	11.27	6.09	3.81	.738	4,220	9.55	670	2.66	1,020	4.08	1,055	1.10
	B	49.42	32.37	11.77	6.42	9.27	.697	4,260	10.59						
	Center	54.42	29.02	10.70	5.85	1.12	.773	3,330	13.58	705	2.13	985	.51	1,106	3.12
	C	59.58	25.70	9.40	5.32	11.28	.854	3,740	2.91	690	.24	935	4.59	1,045	2.04
	D	54.62	29.20	10.02	6.15	2.58	.752	3,710	3.68						
Average		54.05	29.35	10.63	5.97	5.61	.763	3,852	8.06	688	1.78	980	3.06	1,067	2.09
Batch variation	per cent	4.79	5.62	6.94	5.10										
No. 29 (Aug. 15)	A	52.45	30.75	10.80	5.99	3.21	.743	2,970	1.07	632	10.30	930	2.96	845	.20
	B	48.12	34.12	11.35	6.40	4.44	.679	3,100	3.26						
	Center	51.78	31.12	10.85	6.24	1.18	.734	2,790	7.06	595	3.84	840	7.01	870	2.75
	C	52.23	30.75	10.57	6.44	2.79	.737	3,010	.27	492	14.14	940	4.06	825	2.56
	D	50.11	32.20	11.17	6.51	1.98	.706	3,140	4.59						
Average		50.94	31.79	10.95	6.32	2.78	.720	3,002	3.25	573	9.43	903	4.68	847	1.84
Batch variation	per cent	2.86	3.45	2.28	2.53										
No. 32 (Aug. 16)	A	52.98	30.05	10.95	6.01	3.20	.747	3,460	13.43	585	6.00	935	4.91	880	11.12
	B	51.75	31.05	10.95	6.25	.97	.728	2,750	9.82						
	Center	48.48	32.62	12.15	6.74	6.62	.681	3,040	.33	612	1.65	950	3.39	1,020	3.03
	C	52.86	30.05	10.92	6.16	2.62	.747	2,910	4.58	670	7.66	1,065	8.31	1,070	8.08
	D	51.45	31.15	10.95	6.45	1.20	.725	3,090	1.31						
Average		51.51	30.98	11.18	6.32	2.92	.726	3,050	5.89	622	5.11	983	5.54	990	7.41
Batch variation	per cent	2.39	2.42	3.44	3.45										
No. 34 (Aug. 18)	A	47.95	31.00	11.90	6.15	2.98	.676	4,020	1.30	755	1.25	980	4.45	965	9.88
	B	48.86	33.25	11.72	6.16	1.60	.691	4,160	2.14						
	Center	48.04	33.62	12.10	6.23	3.49	.680	4,055	.44	795	6.61	895	4.61	835	4.93
	C	50.82	32.00	11.31	5.86	2.47	.723	3,990	2.04	687	7.87	940	.18	835	4.93
	D	52.47	31.00	10.60	5.93	5.32	.744	4,140	1.64						
Average		49.63	32.77	11.52	6.06	3.17	.703	4,073	1.51	746	5.24	938	3.08	878	6.58
Batch variation	per cent	3.25	3.11	3.98	2.34										
No. 38 (Aug. 19)	A	50.31	31.52	12.05	6.11	.66	.715	3,560	3.49	665	4.28	1,025	3.71	905	3.21
	B	47.45	33.85	12.09	6.61	5.85	.667	3,760	9.30						
	Center	50.51	31.37	12.07	6.03	.98	.717	3,380	1.74	643	.83	905	8.43	930	.53
	C	45.51	34.90	12.90	6.68	9.63	.611	3,390	1.45	605	5.13	1,035	4.73	970	3.74
	D	58.75	25.87	10.15	5.22	15.86	.842	3,110	9.59						
Average		50.51	31.50	11.85	6.13	6.60	.716	3,440	5.11	638	3.41	988	5.62	935	2.49
Batch variation	per cent	6.55	7.33	5.76	6.76										
AVERAGES															
	A	50.70	31.64	11.48	6.18	2.91	.717	3,398	4.08	616	6.34	920	4.52	875	4.67
	B	49.82	32.21	11.72	6.24	4.17	.708	3,408	6.61						
	Center	50.75	31.47	11.58	6.19	2.58	.718	3,208	5.18	605	4.03	879	5.36	889	2.71
	C	53.11	29.11	11.02	5.94	5.74	.755	3,224	3.49	598	7.46	905	3.81	891	4.73
	D	52.39	30.55	10.94	6.11	4.23	.741	3,331	4.50						
Grand average		51.35	31.16	11.35	6.13	3.95	.727	3,314	4.77	606	5.94	901	4.56	885	4.04
Batch variation	per cent	3.48	4.04	4.21	4.08										
Design values		53.17	29.97	11.01	5.85		.770					650		650	

These tables give the results of the analysis of each sample, the percentage of coarse aggregate, sand, cement, and water it contained; the average percentage variation of each material within the batch; the variation factor within the sample, and the compressive and transverse strength of the respective cylinders and beams. Column 2 of each of these tables identifies the part of the batch from which the sample was taken as follows: Looking away from the mixer, sample A was taken from the left front corner of the batch, B from the right front corner, that called Center from the central portion of the batch, C from the left rear, and D from the right rear corner.

Under the caption "Proportions by weight" thes tables show in percentage by weight the values give by the analysis for the gravel, sand, cement, an water found in each sample. Each batch has also bee summarized, and then all batches sampled on eac job for a given batch size have been averaged togethe to obtain the summaries shown at the bottom of each of the various tables, and also combined into the smaller summary tables for each job or study.

If it were possible so to proportion and mix the concrete as to obtain the same amount of gravel, sand, cement, and water in all parts of the batch, the percentage variation would, of course, be zero. However,

TABLE 2.—*General summary showing effect of size of batch on uniformity of mix and strength of concrete, for State-aid project 2926, Sheboygan County, Wis.; 27E paver, 1927 model, good condition; aggregates, good limestone gravel, pit sand*

Size of batch	Part of batch	Proportions by weight				Average sample variation factor	Workability factor b/b_0	Cylinders, 28 days		Beams					
										7 days		28 days			
												End break		Center break	
		Gravel	Sand	Cement	Water			Strength	Variation	Strength	Variation	Strength	Variation	Strength	Variation
		Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*		*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*
27 cubic feet	A	50.70	31.64	11.48	6.18	2.91	0.717	3,398	4.08	616	6.34	920	4.52	875	4.67
	B	49.82	32.21	11.72	6.24	4.17	.708	3,408	6.61						
	Center	50.75	31.47	11.58	6.19	2.58	.718	3,208	5.18	605	4.03	879	5.36	889	2.71
	C	53.11	29.11	11.02	5.94	5.74	.755	3,224	3.49	598	7.46	905	3.81	891	4.73
	D	52.39	30.55	10.94	6.11	4.23	.741	3,331	4.50						
Average		51.35	31.16	11.35	6.13	3.95	.727	3,314	4.77	606	5.94	901	4.56	885	4.04
Batch variation	per cent	3.48	4.04	4.21	4.08										
30 cubic feet	A	50.05	32.34	11.28	6.31	5.32	.708	3,252	7.13	582	5.37	868	4.12	818	4.66
	B	49.21	32.62	11.67	6.49	6.67	.692	3,433	6.23						
	Center	52.11	30.82	10.83	6.22	2.57	.739	3,262	4.33	586	2.06	855	3.90	870	4.78
	C	56.56	27.76	10.15	5.52	8.51	.810	3,178	8.99	568	5.45	859	2.74	882	6.03
	D	54.21	29.36	10.57	5.86	4.03	.771	3,372	9.21						
Average		52.43	30.58	10.90	6.08	5.42	.744	3,300	7.20	579	4.25	861	3.59	856	5.17
Batch variation	per cent	4.77	5.69	5.70	5.52										
33 cubic feet	A	49.97	31.87	11.92	6.24	3.78	.708	3,508	6.39	612	4.22	861	5.66	868	5.02
	B	49.80	31.98	11.94	6.28	5.02	.704	3,433	6.16						
	Center	51.93	30.30	11.65	6.10	4.45	.737	3,539	5.18	642	3.64	920	4.05	882	5.03
	C	54.60	28.51	10.94	5.96	6.90	.776	3,474	3.98	636	3.49	919	3.18	951	8.02
	D	52.50	30.23	11.21	6.04	4.65	.745	3,608	6.60						
Average		51.76	30.58	11.53	6.12	4.96	.734	3,512	5.66	630	3.78	901	4.29	900	6.03
Batch variation	per cent	4.41	5.04	5.46	4.94										
35 cubic feet	A	50.99	31.63	11.25	6.12	2.76	.716	3,148	4.94	602	2.91	882	5.21	897	5.04
	B	51.21	31.53	11.09	6.16	3.59	.732	3,102	6.66						
	Center	49.85	32.14	11.67	6.33	5.17	.705	3,359	5.77	607	3.73	886	3.14	903	5.53
	C	53.23	30.29	10.53	5.93	5.80	.756	3,069	6.20	611	3.17	861	3.35	854	3.95
	D	54.08	29.57	10.63	5.71	5.66	.770	3,211	8.79						
Average		51.88	31.03	11.04	6.05	4.59	.735	3,178	6.47	607	3.27	876	3.90	885	4.84
Batch variation	per cent	3.97	4.06	5.99	4.37										

TABLE 3.—*General summary showing effect of size of batch on uniformity of mix and strength of concrete, for State-aid project 2926, Sheboygan County, Wis.; 27E paver, new; aggregate, good limestone gravel, pit sand*

Size of batch	Part of batch	Proportions by weight				Average sample variation factor	Workability factor b/b_0	Cylinders, 28 days		Beams					
										7 days		28 days			
												End break		Center break	
		Gravel	Sand	Cement	Water			Strength	Variation	Strength	Variation	Strength	Variation	Strength	Variation
		Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*		*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in*	*Per cent*	*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*
27 cubic feet	A	52.89	29.63	11.51	5.96	3.65	0.753	3,616	10.29	613	5.45	890	3.80	859	4.39
	B	52.76	29.95	11.55	5.73	3.07	.751	3,491	5.40						
	Center	53.79	29.07	11.36	5.77	3.69	.766	3,370	8.21	573	3.95	837	4.56	824	4.81
	C	52.78	29.76	11.59	5.87	2.72	.751	3,623	4.60	593	2.76	878	2.71	880	5.56
	D	52.78	29.82	11.60	5.81	4.24	.752	3,671	8.90						
Average		53.00	29.65	11.52	5.83	3.48	.755	3,554	7.48	593	4.05	868	3.69	854	4.92
Batch variation	per cent	2.59	3.40	4.14	3.77										
30 cubic feet	A	52.13	30.07	11.88	5.92	3.36	.740	3,678	8.91	611	5.34	913	4.48	903	7.33
	B	53.38	29.34	11.59	5.68	2.69	.762	3,711	4.99						
	Center	53.93	28.70	11.62	5.74	4.69	.769	3,494	4.47	619	3.83	919	3.58	921	4.81
	C	53.35	29.04	11.66	5.94	4.98	.760	3,577	9.96	614	4.05	873	4.82	890	4.89
	D	51.43	30.58	12.01	5.98	4.17	.728	3,729	5.39						
Average		52.84	29.55	11.75	5.85	3.98	.752	3,658	6.75	615	4.41	901	4.29	904	5.64
Batch variation	per cent	3.18	3.72	3.50	5.52										
33 cubic feet	A	51.87	30.69	11.53	5.91	2.36	.736	3,911	6.29	623	4.20	905	4.10	864	4.63
	B	52.65	30.18	11.26	5.91	4.00	.748	3,956	5.70						
	Center	52.44	30.02	11.59	5.94	4.31	.744	3,819	4.86	609	3.58	906	2.72	900	4.13
	C	50.80	31.29	11.92	5.44	4.03	.721	3,947	5.32	624	5.77	914	3.77	864	4.91
	D	52.42	30.65	11.16	5.10	3.19	.747	3,874	6.89						
Average		52.04	30.56	11.49	5.90	3.58	.739	3,901	5.81	619	4.49	908	3.53	876	4.56
Batch variation	per cent	2.87	3.66	4.20	3.59										
35 cubic feet	A	51.65	30.85	11.69	5.81	2.70	.735	3,858	4.83	610	6.83	916	4.80	913	3.86
	B	52.58	30.27	11.43	5.71	3.23	.749	4,025	6.36						
	Center	51.69	30.57	11.87	5.86	3.31	.736	3,809	5.89	600	6.11	945	4.88	870	5.15
	C	53.01	29.76	11.38	5.85	4.69	.753	3,818	4.77	611	3.19	910	5.83	936	4.62
	D	53.52	29.48	11.40	5.58	4.75	.763	3,719	7.34						
Average		52.50	30.18	11.55	5.77	3.74	.748	3,846	5.84	607	5.34	922	5.17	903	4.54
Batch variation	per cent	2.92	3.74	3.89	4.39										

since it is largely by chance that the individual particles which make up the concrete go where they do, there will be variations in the distribution of some or all of the ingredients. The greater this variation, the less will be the uniformity of samples taken from point to point within a batch. To compare the amount of variation within a batch, a factor of variation, or percentage variation factor, has been computed and is shown in these tables for each individual sample. This variation factor has been computed as follows: The percentages of gravel, sand, cement, and water for the five parts of each separate batch have been averaged and the arithmetical average of the percentage variation of each of the four ingredients in a sample from the average of the like ingredients for the batch, when expressed as a percentage, equals the variation factor for the sample. This factor is given in the seventh column of Table 1. The larger the number the greater the variation, and the smaller the number the greater the apparent uniformity. The percentage variation of each ingredient from the average of the batch is shown below the average of the percentage by weight of the four ingredients (gravel, sand, cement, and water) and was computed for each batch and also for the summary sheets.

The workability factor, b/b_0, tabulated in the eighth column, is the ratio which the solid or absolute volume of the coarse aggregates in a cubic foot of the concrete bears to the solid or absolute volume of a cubic foot of the coarse aggregate. The percentage of excess mortar is equal to $1-b/b_0$. For any given set of job conditions, this factor appears to be a good index of how workable the batch will be. Under the heading "Cylinders, 28 days" is shown the compressive strength for each cylinder as well as the average of all cylinders made from each batch. These data are so arranged as to correspond to the other factors shown for the same parts of the batch. The beam strengths are shown in like manner.

On the Arkansas and Wisconsin projects definite amounts of cement and water were specified and the grading of the coarse aggregate was closely controlled by the use of separate sizes. The mix was designed entirely by State engineers. Comparable conditions were therefore obtained throughout the tests, and the same materials were used for each series of tests.

SIZE OF BATCH SHOWS NO ESSENTIAL INFLUENCE ON CONCRETE QUALITY

Tables 2, 3, 4, and 5 give a concise summary of the principal part of the work on the two Wisconsin jobs. An examination of the values given in these tables for the average variations of the concrete ingredients within the batch, the average compressive and transverse strengths, and the average percentage variations of these strengths within a batch, indicates that with a 60-second setting of the batchmeter no essential difference can be detected between a 27-cubic-foot batch and a 35-cubic-foot batch for any of the four mixers used on those two jobs. For example, if we combine the work on the two Wisconsin jobs for the period during which the new mixers were used, as this would illustrate the results which can be obtained under the most favorable conditions, we have the values given in Table 6.

If we now turn to the period when 3-year-old mixers were used on these two jobs as representing the re-

TABLE 4.—*General summary showing effect of size of batch on uniformity of mix and strength of concrete, for State-aid project 2916, Sheboygan County, Wis.; 27E paver, 1927 model, good condition; aggregates, good limestone gravel, pit sand*

Size of batch	Part of batch	Proportions by weight				Average sample variation factor	Workability factor b/b_0	Cylinders, 28 days		Beams					
										7 days		28 days			
												End break		Center break	
		Gravel	Sand	Cement	Water			Strength	Variation	Strength	Variation	Strength	Variation	Strength	Variation
		Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*		*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*
27 cubic feet	A	50.28	31.79	11.16	6.09	6.33	0.716	3,726	5.96	578	4.48	900	4.57	887	3.19
	B	50.87	31.28	10.89	6.29	6.06	.728	3,572	5.97						
	Center.	55.99	27.58	10.06	5.69	7.10	.803	3,340	9.58	590	7.23	890	5.34	872	1.77
	C	53.47	29.21	10.82	5.84	5.17	.770	3,557	4.98	567	4.86	879	3.46	825	3.90
	D	53.75	29.28	10.50	5.79	6.45	.773	3,550	4.99						
Average		52.87	29.83	10.68	5.94	6.22	.760	3,549	6.30	578	5.52	890	4.46	865	2.88
Batch variation	per cent	5.07	6.45	6.94	6.42										
30 cubic feet	A	49.89	31.46	11.89	6.00	4.60	.710	3,803	6.17	631	3.48	926	4.55	884	7.15
	B	50.74	31.19	11.12	6.14	3.43	.732	3,863	7.13						
	Center.	53.64	29.38	10.46	5.69	5.96	.783	3,500	8.20	633	3.31	938	3.22	886	6.32
	C	53.61	29.34	10.42	5.89	5.78	.788	3,821	5.44	619	4.50	915	4.01	905	5.06
	D	50.81	31.18	11.19	6.02	3.44	.738	3,678	5.48						
Average		51.73	30.50	11.01	5.94	4.62	.749	3,733	6.48	628	3.74	927	3.92	890	6.35
Batch variation	per cent	3.62	4.30	6.60	3.94										
33 cubic feet	A	50.68	31.00	11.45	6.16	4.48	.724	3,653	3.73	603	4.55	893	4.46	896	3.45
	B	50.44	31.43	11.16	6.21	3.41	.721	3,614	6.02						
	Center.	53.09	29.77	10.48	5.91	4.49	.765	3,568	3.97	595	3.63	896	6.11	882	3.77
	C	51.74	30.37	11.13	6.05	3.55	.738	3,823	4.40	617	4.40	940	5.64	899	3.03
	D	51.95	30.57	10.84	5.94	3.88	.746	3,742	4.12						
Average		51.59	30.63	11.01	6.05	3.96	.738	3,680	4.45	605	4.20	910	5.40	892	3.42
Batch variation	per cent	2.97	3.16	5.88	3.85										
35 cubic feet	A	51.18	31.37	10.88	5.89	5.51	.735	4,015	3.74	600	3.37	936	3.74	919	3.82
	B	51.50	30.89	10.91	6.04	3.76	.737	3,962	5.03						
	Center.	50.73	31.51	11.44	5.66	4.09	.729	3,827	3.39	613	5.01	925	5.93	899	2.81
	C	54.01	29.28	10.25	5.74	6.06	.777	3,846	5.59	597	3.95	930	6.61	916	2.98
	D	52.79	29.88	11.18	5.47	4.67	.762	3,865	8.71						
Average		52.00	30.61	10.95	5.76	4.79	.747	3,903	5.29	603	4.11	930	5.43	913	3.22
Batch variation	per cent	3.03	3.58	6.50	6.05										

TABLE 5.—*General summary showing effect of size of batch on uniformity of mix and strength of concrete, for State-aid project 2916, Sheboygan County, Wis.; 27–E paver, new; aggregates, good limestone gravel, pit sand*

Size of batch	Part of batch	Proportions by weight				Average sample variation factor	Workability factor b/b_0	Cylinders, 28 days		Beams, 7 days		Beams, 28 days, End break		Beams, 28 days, Center break	
		Gravel	Sand	Cement	Water			Strength	Variation	Strength	Variation	Strength	Variation	Strength	Variation
		Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*		*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*
27 cubic feet	A	51.31	30.95	11.85	5.56	4.27	0.739	3,632	5.71	574	5.88	853	3.17	832	5.67
	B	51.54	30.82	11.69	5.56	3.46	.741	3,601	4.75						
	Center	54.24	28.94	11.03	5.41	4.68	.779	3,480	6.15	570	6.11	810	5.29	842	4.87
	C	52.98	29.56	11.61	5.46	4.33	.766	3,708	7.37	592	4.24	845	5.24	877	6.54
	D	52.10	30.19	11.87	5.53	3.50	.750	3,748	6.24						
Average		52.43	30.09	11.61	5.51	4.05	.755	3,634	6.04	579	5.41	836	4.57	850	5.69
Batch variation	per cent	3.19	4.39	4.49	4.13										
30 cubic feet	A	50.85	31.08	12.23	5.55	3.57	.733	3,660	4.10	586	4.47	846	5.15	873	6.62
	B	51.45	30.66	12.14	5.48	3.26	.743	3,664	4.21						
	Center	52.10	30.33	11.79	5.48	3.06	.751	3,569	3.44	581	4.40	831	4.70	875	5.79
	C	51.95	30.40	11.95	5.45	3.89	.752	3,809	3.63	598	5.64	857	6.53	857	6.13
	D	50.66	31.10	12.58	5.36	4.46	.732	3,679	6.19						
Average		51.40	30.71	12.14	5.46	3.65	.744	3,676	4.31	588	4.84	845	5.46	869	6.18
Batch variation	per cent	2.38	2.96	5.17	4.08										
33 cubic feet	A	52.38	30.06	11.38	5.72	3.03	.755	3,496	5.77	592	5.79	856	5.17	853	5.69
	B	51.64	30.82	11.54	5.65	4.07	.753	3,488	3.51						
	Center	52.69	29.72	11.66	5.60	3.59	.758	3,277	8.08	588	4.61	851	4.65	851	5.97
	C	53.50	29.00	11.34	5.75	5.11	.767	3,550	5.59	590	3.44	854	7.37	844	7.66
	D	53.17	29.56	11.37	5.49	3.27	.767	3,541	9.14						
Average		52.68	29.83	11.46	5.64	3.81	.757	3,470	6.42	590	4.65	854	5.61	852	6.40
Batch variation	per cent	2.91	3.97	4.48	3.91										
35 cubic feet	A	50.98	31.20	11.81	5.61	3.66	.732	3,648	5.83	586	5.67	830	6.80	825	5.08
	B	51.80	31.15	10.97	5.68	3.93	.742	3,862	7.12						
	Center	51.95	30.52	11.70	5.40	3.79	.750	3,662	4.23	606	5.50	846	8.00	815	4.72
	C	53.97	29.38	10.82	5.46	4.62	.778	3,806	4.58	584	4.35	798	6.06	795	6.77
	D	52.00	30.41	11.53	5.68	3.44	.747	3,686	8.19						
Average		52.14	30.53	11.37	5.56	3.89	.749	3,733	5.99	592	5.17	825	6.95	812	5.48
Batch variation	per cent	2.55	3.15	5.30	4.56										

sults which might be expected when old equipment is used, we have the values given in Table 7.

TABLE 6.—*Combined average values of compressive strength, modulus of rupture, and variation factor obtained on Wisconsin State-aid projects 2916 and 2926 for periods during which new mixers were used*

Size of batch	Compressive strength of cylinders at 28 days	Modulus of rupture at 28 days	Average variation		Variation factor within batch
			Cylinders	Beams	
	Lbs. per sq. in.	*Lbs. per sq. in.*	*Per cent*	*Per cent*	*Per cent*
27 cubic feet	3,594	852	6.76	4.72	3.76
30 cubic feet	3,667	880	5.53	5.39	3.81
33 cubic feet	3,685	872	6.11	5.02	3.70
35 cubic feet	3,790	866	5.92	5.53	3.81
General average	3,684	867	6.08	5.16	3.77

TABLE 7.—*Combined average values of compressive strength, modulus of rupture, and variation factor obtained on Wisconsin State-aid projects 2916 and 2926 for periods during which old mixers were used*

Size of batch	Compressive strength of cylinders at 28 days	Modulus of rupture at 28 days	Average variation		Variation factor within batch
			Cylinders	Beams	
	Lbs. per sq. in.	*Lbs. per sq. in.*	*Per cent*	*Per cent*	*Per cent*
27 cubic feet	3,431	885	5.53	3.98	5.08
30 cubic feet	3,516	883	6.84	4.76	5.02
33 cubic feet	3,596	901	5.05	4.78	4.46
35 cubic feet	3,540	901	5.88	4.35	4.69
General average	3,518	892	5.82	4.47	4.81

We find nothing here that indicates any significant difference. For the period during which the old mixers were used the cylinder strengths average a trifle lower, but the beams were stronger, and, while the batch variation was a little higher for the old mixers, because of spreader-bucket design, the variation of both beam and cylinder strengths was less. The value of the concrete produced by the old mixers, therefore, seems equal to that produced by the new mixers, but, as will be shown later, the rate at which concrete could be produced was considerably greater for the new mixers.

Analyzing the summary table for Arkansas (Table 8) for the same variation factors as to uniformity and strength, we find that these data indicate that the quality of concrete was not appreciably affected by the use of batches varying from 27 to 37 cubic feet.

INCREASE IN BATCHMETER SETTING ABOVE 50 SECONDS PRODUCED LITTLE EFFECT

In order to investigate further the effect of the mixing time when larger than normal batches are used the setting of the batchmeter was varied to give mixing times of 50, 60, and 80 seconds during one series of tests on the Wisconsin jobs. Both 30 and 33 cubic-foot batches were tested in this manner. These data are summarized in Tables 9 and 10, for the 30 and 33 cubic-foot batches, respectively. Average values for both sizes of batch are given in Table 11. As shown in this outline, mixing either the 30 or the 33 cubic-foot batch more than 50 seconds (batchmeter setting) did not produce higher strength and resulted in only a very slight apparent improvement in uniformity of mixing.

Further light on the variations in uniformity and strength of the concrete on the several jobs under discussion may be obtained from a study of the data given in Table 12. The detailed tables, of which Table 1 is

TABLE 8.—*General summary showing effect of size of batch on uniformity of mix and strength of concrete, for Federal-aid project 259–A, Jefferson County, Ark., 27E paver, good condition; aggregates, trap rock, river sand*

Size of batch	Part of batch	Proportions by weight				Average sample variation factor	Workability factor b/b_0	Cylinders, 28 days		Beams					
										7 days		28 days			
												End break		Center break	
		Stone	Sand	Cement	Water			Strength	Variation	Strength	Variation	Strength	Variation	Strength	Variation
		Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*		*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*
27 cubic feet	A	47.58	33.56	12.08	6.53	5.62	0.733	4,548	5.76	721	9.36	705	8.04	715	4.37
	B	49.66	32.27	11.83	6.05	3.92	.771								
	Center	49.33	32.48	11.90	6.06	4.38	.767			627	5.23	681	6.14	716	5.22
	C	50.61	31.78	11.27	6.08	6.35	.783	4,594	5.76	637	7.06	735	7.37	683	5.80
	D	52.74	29.72	11.53	5.76	5.06	.818								
Average		49.90	32.02	11.73	6.11	5.07	.773	4,571	5.76	662	7.22	707	7.18	705	5.13
Batch variation	per cent	3.97	4.35	5.44	6.51										
30 cubic feet	A	46.43	34.57	12.39	6.35	4.87	.716	4,526	7.00	641	5.81	693	6.64	677	6.13
	B	48.17	33.22	12.20	6.15	5.15	.750								
	Center	48.65	33.22	11.73	6.15	5.19	.752			641	6.84	680	8.66	692	6.77
	C	48.24	33.14	12.61	5.77	6.08	.751	4,555	7.00	633	6.37	712	4.25	686	3.22
	D	50.38	32.35	11.07	5.96	6.27	.779								
Average		48.37	33.30	12.00	6.08	5.51	.749	4,540	7.00	638	6.34	695	6.52	685	5.37
Batch variation	per cent	4.74	4.44	8.45	4.42										
32 cubic feet	A	46.78	34.88	11.85	6.24	3.90	.721	5,327	5.15	689	7.77	745	3.77	695	3.38
	B	49.52	32.84	11.35	6.04	4.57	.767								
	Center	47.36	34.37	11.86	6.16	3.94	.731			678	4.38	750	4.62	704	4.53
	C	47.14	34.52	12.05	6.05	6.30	.733	5,579	5.15	658	8.39	717	2.95	714	4.25
	D	48.56	33.71	11.61	5.88	5.18	.752								
Average		47.87	34.06	11.74	6.07	4.78	.741	5,453	5.15	675	6.85	737	3.78	704	4.05
Batch variation	per cent	4.14	3.97	6.65	4.35										
34 cubic feet	A	48.04	33.96	11.57	6.18	4.71	.742	5,596	4.83	761	6.09	753	3.59	756	8.05
	B	47.65	34.41	11.81	5.88	3.14	.732								
	Center	47.80	34.32	11.71	5.92	3.07	.742			719	5.55	774	6.07	696	5.48
	C	49.46	33.01	11.33	5.94	4.58	.766	5,534	4.83	671	7.72	718	6.33	672	6.70
	D	49.13	33.13	11.68	5.80	4.18	.764								
Average		48.39	33.79	11.62	5.95	3.92	.749	5,565	4.83	717	6.45	748	5.33	708	6.74
Batch variation	per cent	3.14	3.26	5.29	4.00										
37 cubic feet	A	47.30		12.15	6.11	5.36	.739	4,971	6.68	681	3.98	705	7.15	676	5.77
	B	48.69	33.72	11.30	6.04	3.71	.752								
	Center	47.64	34.07	12.23	5.82	3.62	.738			702	7.22	718	7.53	695	5.35
	C	50.20	32.47	11.58	5.51	5.21	.775	5,060	6.68	667	6.88	758	7.86	690	7.14
	D	48.16	34.03	11.73	5.83	4.53	.731								
Average		48.40	33.70	11.80	5.86	4.49	.747	5,016	6.68	683	6.03	726	7.50	687	6.08
Batch variation	per cent	3.30	2.94	6.61	5.10										
General average						4.75	.752	5,029	5.88	675	6.58	722	6.06	698	5.47

an example, were examined to determine, for each batch on all jobs, the maximum and minimum values of variation factor, compressive strength and percentage variation, and flexural strength and percentage variation. These values were then averaged for each job on the basis of batch size and mixing time. The results of this computation are given in Table 12. A final condensed summary of average values for each of the three jobs is given in Table 13.

It is interesting to note that the average job values of the cement content and water content on the Wisconsin and Arkansas jobs were very nearly equal. The specifications for the Wisconsin jobs called for a minimum cement content of 5 sacks per cubic yard of concrete and 6 gallons of water per sack of cement; but as the work progressed, it was found that less water was required for workability than was anticipated, with the result that a cement content of slightly more than 5 sacks per cubic yard was obtained. The specifications for the Arkansas project required a minimum cement content of 5.40 sacks of cement per cubic yard of concrete and 5.25 gallons of water per sack of cement; but, because of the extremely high temperatures and low humidity experienced during much of the time of this study, more water was required than the engineer had anticipated. The coarse aggregate used on the Arkansas job was a crushed trap rock, while the coarse aggregate used on the Wisconsin jobs was a limestone gravel. The maximum size of aggregate on all three jobs was approximately 2½ inches.

Except in the case of the two Wisconsin jobs, both of which used the same brand of cement and the same kinds of aggregate, the beam and cylinder strengths are not really comparable one job with another, because different kinds of aggregates and cement were used on the other job.

With these qualifications in mind, it may be said for the three jobs studied that the data on uniformity of mixing and resultant strength indicate that as good concrete can be produced from batches at least as large as 34 cubic feet as from batches of any smaller size between 27 and 34 cubic feet, when all solid materials are mixed 55 seconds (i. e., with a batchmeter setting of 60 seconds). They also indicate that where State or county engineers control the mix, the size of the batch can be at least as large as 33 cubic feet and still produce equally good concrete with a batchmeter setting of 50 seconds, or an actual mixing time of 45 seconds for all solid materials in standard 27E mixers with the blades and buckets in fair condition.

Another point noted in the course of this work was that when the amount of water per cubic yard of concrete was less than 28 gallons the mix was so dry that the finishing operations were likely to be delayed.

TABLE 9.—*General summary showing effect of size of batch and length of mixing time on uniformity of mix and strength of concrete mixed in batches of 30 cubic feet on State-aid project 2926, Sheboygan County, Wis.; 27E paver, new; aggregates, good limestone gravel, pit sand*

Mixing time	Part of batch	Proportions by weight				Average sample variation factor	Workability factor b/b_0	Cylinders, 28 days		Beams					
										7 days		28 days			
												End break		Center break	
		Gravel	Sand	Cement	Water			Strength	Variation	Strength	Variation	Strength	Variation	Strength	Variation
		Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*		*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*
50 seconds	A	53.29	29.79	10.95	5.97	3.58	0.757	3,804	7.52	570	6.17	801	5.43	794	4.99
	B	51.98	30.40	11.65	5.97	3.24	.738	3,683	6.33						
	Center	53.34	29.37	11.18	6.10	2.76	.754	3,787	3.27	561	7.21	875	3.62	844	3.07
	C	52.01	30.40	11.40	6.18	2.76	.734	3,893	6.51	587	8.04	867	3.07	865	4.20
	D	52.35	30.30	11.19	6.15	2.87	.736	4,119	6.50						
Average		52.59	30.05	11.28	6.08	3.04	.744	3,857	6.03	573	7.14	847	4.04	835	4.09
Batch variation	per cent	2.61	2.91	3.37	3.28										
60 seconds	A	52.45	30.55	11.07	5.92	2.37	.741	4,007	5.36	576	6.28	857	4.96	815	4.06
	B	52.07	30.62	11.28	6.03	3.25	.736	3,940	7.70						
	Center	55.61	28.06	10.58	5.74	5.01	.790	3,557	8.86	587	3.80	825	3.86	837	3.16
	C	53.22	29.63	11.11	6.03	3.02	.752	3,899	8.89	575	3.80	866	4.11	875	3.78
	D	53.08	30.06	10.82	6.03	3.08	.751	3,773	5.23						
Average		53.29	29.78	10.97	5.96	3.35	.754	3,835	7.21	597	4.63	849	4.31	842	3.67
Batch variation	per cent	2.80	3.59	2.87	4.19										
80 seconds	A	52.36	30.21	11.33	6.12	3.76	.742	3,906	5.48	579	3.62	809	2.18	818	5.62
	B	52.26	30.17	11.42	6.14	4.19	.740	3,826	3.65						
	Center	54.56	28.66	10.82	5.95	4.73	.775	3,762	4.00	566	3.32	806	4.00	819	3.11
	C	52.71	29.96	11.15	6.17	3.38	.747	3,844	3.90	569	1.61	819	3.31	840	3.85
	D	51.77	30.60	11.40	6.23	2.41	.733	3,907	5.45						
Average		52.73	29.92	11.22	6.13	3.69	.747	3,849	4.49	571	2.85	811	3.16	826	4.19
Batch variation	per cent	3.17	3.68	3.86	4.04										

TABLE 10.—*General summary showing effect of size of batch and length of mixing time on uniformity of mix and strength of concrete mixed in batches of 30 cubic feet on State-aid project 2926, Sheboygan County, Wis.; 27E paver, new; aggregates, good limestone gravel, pit sand*

Mixing time	Part of batch	Proportions by weight				Average sample variation factor	Workability factor b/b_0	Cylinders, 28 days		Beams					
										7 days		28 days			
												End break		Center break	
		Gravel	Sand	Cement	Water			Strength	Variation	Strength	Variation	Strength	Variation	Strength	Variation
		Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*		*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*
50 seconds	A	52.47	29.73	11.73	6.07	2.38	0.743	4,080	6.69	590	2.71	859	2.61	855	4.16
	B	53.00	29.61	11.40	5.97	3.81	.752	3,857	7.18						
	Center	52.53	29.65	11.71	6.10	2.96	.744	3,611	6.28	617	4.75	833	4.28	838	2.96
	C	52.19	30.35	11.23	6.22	3.35	.737	3,703	6.94	596	4.93	840	4.53	785	4.89
	D	52.52	30.06	11.35	6.06	3.52	.743	4,026	5.43						
Average		52.54	29.88	11.48	6.09	3.20	.744	3,855	6.50	601	4.13	844	3.81	826	4.06
Batch variation	per cent	2.51	3.23	3.47	3.61										
60 seconds	A	54.30	28.79	11.15	5.76	2.03	.773	4,049	12.11	580	4.91	839	3.15	852	4.35
	B	53.68	29.13	11.39	5.79	2.67	.765	3,983	5.98						
	Center	54.97	28.14	11.03	5.84	3.60	.780	3,654	7.10	590	3.64	836	5.48	862	3.60
	C	53.41	29.34	11.21	6.03	4.42	.758	3,759	7.02	574	4.32	822	3.33	828	5.80
	D	53.44	29.34	11.30	5.91	2.17	.760	3,740	5.67						
Average		53.93	28.95	11.21	5.87	2.98	.767	3,837	7.58	581	4.29	832	4.00	847	4.59
Batch variation	per cent	2.35	3.11	2.59	3.85										
80 seconds	A	52.94	29.69	11.44	5.92	3.63	.753	4,090	8.81	594	4.47	822	4.85	839	4.05
	B	52.95	29.39	11.69	5.95	3.59	.752	4,014	6.83						
	Center	53.66	29.22	11.17	5.95	2.68	.762	3,518	6.95	571	6.73	839	4.70	884	7.33
	C	54.99	28.13	11.03	5.84	5.31	.782	3,579	7.33	580	4.83	826	2.01	948	5.95
	D	53.68	29.10	11.42	5.80	5.15	.765	3,603	4.60						
Average		53.65	29.11	11.35	5.89	1.07	.763	3,761	6.90	582	5.34	829	3.85	843	5.70
Batch variation	per cent	3.55	4.38	4.36	4.01										

Furthermore, workability ratios (b/b_0) above 0.85 for gravel and 0.75 for stone proved to be in general undesirable, in that there was too much coarse aggregate in the mix for the present method of placing and finishing, a condition which tends to delay the finishing operations. The data also seem to indicate that there is a fairly close relationship between the cement and water contents and the resulting compressive and transverse strengths.

Variations within a batch can not be entirely eliminated by mechanical mixing, no matter what the size of the batch or the length of the mixer cycle. That this is so becomes apparent when one considers that the distribution of the particles of aggregate, cement,

TABLE 11.—*Effect of size of batch and length of mixing time on uniformity of mix and strength of concrete mixed in batches of 30 and 33 cubic feet on State-aid project 2926, Sheboygan County, Wis.*

Mixing time	Batch size	Compressive strength of cylinders at 28 days	Average variation		
			Modulus of rupture at 28 days	Cylinders	Beams	Variation factor within batch
	Cubic feet	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Per cent*	*Per cent*	*Per cent*
50 seconds	30	3,857	841	6.03	4.07	3.04
	33	3,855	835	6.50	3.94	3.20
60 seconds	30	3,835	846	7.21	3.99	3.35
	33	3,837	840	7.58	4.30	2.98
80 seconds	30	3,849	818	4.49	3.67	3.69
	33	3,761	836	6.90	4.78	4.07
General average	30	3,847	835	5.91	3.91	3.36
	33	3,818	837	6.99	4.34	3.42
Both		3,832	836	6.45	4.12	3.39

FIGURE 5.—THE LAST CUBIC FOOT OF CONCRETE DISCHARGED FROM ANY MAKE OF MIXER HAS A PREDOMINANCE OF COARSE AGGREGATE. THE DESIGN OF MIXER SPREADER-BUCKETS ALSO AFFECTS SEGREGATION. THE RESULTS OF THESE TWO EFFECTS CAUSED A BATCH TO APPEAR AS SHOWN

and water within the batch is very largely a matter of chance. Consequently, a point is soon reached at which the mixing action during each instant displaces as many particles from their proper positions as are rightly placed. After this point is reached, further mixing is evidently a useless expense in so far as securing greater uniformity is concerned.

Figure 5 shows a badly segregated batch. Though this is not a typical condition, the photograph is presented to point out the extent to which variation sometimes takes place within a batch. This variation is largely present as the batch is discharged from the mixer drum, and then a further segregation seems to occur while the concrete is discharged from the spreader bucket. Mixer manufacturers each year have been improving the spreader bucket from the typical box-shaped bucket of a few years ago to the present oblong bucket designed to eliminate a greater part of this further segregation. The two types of bucket are shown in Figure 6.

In all pavers which have been observed, not only on these jobs but also on hundreds of other jobs, the last one-half to 1 cubic foot of material discharged from the mixer drum has a predominance of coarse aggregate. It is recommended, therefore, that during continuous operation about a cubic foot of concrete always be retained in the drum to eliminate this trouble.

In Table 14 values of the variation factor within batch and of the variation in strength of cylinders are tabulated for each batch on Wisconsin State-aid project 2926. The data are arranged to bring out any differences in the magnitude of these variation factors which may be traceable to the use of varying sizes of batch

TABLE 12.—*Values of maximum and minimum variation factor, compressive strength, and flexural strength for each batch, averaged for each job on the basis of batch size and mixing time*

Job	Batch size	Mixing time	Variation factor within batch		Cylinders at 28 days				Beams at 28 days, end break				Beams at 28 days, center break			
					Strength		Variation		Strength		Variation		Strength		Variation	
			Maximum	Minimum	Maximum	Minimum	Maximum	Minimum	Maximum	Minimum	Maximum	Minimum	Maximum	Minimum	Maximum	Minimum
	Cu. ft.	*Seconds*	*Per cent*	*Per cent*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Per cent*	*Per cent*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Per cent*	*Per cent*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Per cent*	*Per cent*
Wisconsin State-aid project 2926, old paver	27	60	7.71	1.40	3,554	3,053	8.39	1.25	954	851	6.84	2.24	932	846	6.05	2.06
	30	60	9.72	1.99	3,733	2,921	14.38	2.23	901	819	5.38	1.21	908	799	7.49	3.33
	33	60	8.79	1.92	3,823	3,203	10.35	2.07	950	846	6.44	1.24	976	840	9.04	3.27
	35	60	9.06	1.66	3,488	2,751	14.20	1.91	921	833	5.85	1.57	946	826	7.26	1.37
Wisconsin State-aid project 2926, new paver	27	60	5.47	1.60	4,029	3,151	15.83	1.40	911	824	5.54	1.20	909	806	7.38	2.49
	30	60	7.75	1.16	4,086	3,230	13.59	1.35	961	846	6.44	1.80	968	849	7.90	3.19
	33	60	6.26	1.44	4,269	3,545	10.38	1.10	952	871	5.29	1.68	928	826	6.83	2.29
	35	60	7.13	1.39	4,215	3,471	11.44	1.31	982	863	7.75	2.47	951	850	6.82	2.41
Wisconsin State-aid project 2916, old paver	27	60	10.87	2.66	3,942	3,187	12.76	1.69	947	844	6.68	1.83	901	815	3.60	1.57
	30	60	9.11	1.83	4,090	3,312	11.88	1.56	968	876	5.69	1.92	935	840	7.94	5.32
	33	60	6.34	1.86	3,967	3,415	8.99	1.03	967	840	8.11	2.26	932	852	5.13	1.25
	35	60	8.16	2.22	4,245	3,570	10.38	1.28	993	865	8.14	2.39	949	877	4.51	1.82
Wisconsin State-aid project 2916, new paver	27	60	6.28	2.02	3,942	3,228	10.87	2.10	883	784	6.85	1.91	919	792	8.54	2.43
	30	60	5.75	1.93	3,932	3,380	8.84	.56	902	786	8.19	2.66	927	792	9.28	2.91
	33	60	6.60	1.60	3,859	3,152	11.52	2.35	908	790	7.86	2.74	915	781	9.20	2.76
	35	60	6.19	2.03	4,122	3,356	12.72	1.98	906	769	9.71	4.31	869	759	7.95	2.41
Wisconsin State-aid project 2926, new paver	30	50	4.96	1.39	4,262	3,490	10.63	2.18	886	799	6.06	1.79	876	792	6.13	2.03
	30	60	5.70	1.41	4,296	3,368	12.59	1.67	899	803	6.46	1.93	881	799	5.50	1.64
	30	80	6.00	1.92	4,097	3,558	8.69	1.46	846	779	4.75	1.32	871	775	6.29	.96
Wisconsin State-aid project 2926, new paver	33	50	5.45	1.71	4,271	3,499	11.01	1.98	889	804	5.71	1.33	871	784	5.80	1.19
	33	60	5.30	1.14	4,389	3,358	15.29	1.79	876	785	5.99	1.45	889	798	6.88	2.35
	33	80	7.22	1.34	4,147	3,319	13.33	1.67	873	781	5.78	1.40	896	776	8.14	2.33
Federal-aid project 259-A, Jefferson County, Ark	27	60	7.74	2.67	4,831	4,311	5.76	5.76	775	655	10.78	4.50	755	654	7.56	1.62
	30	60	9.27	3.00	4,832	4,249	7.00	7.00	751	637	9.77	2.63	733	637	8.06	2.18
	32	60	7.67	1.96	5,736	3,170	5.15	5.15	775	706	5.67	1.54	743	668	6.08	1.63
	34	60	6.70	2.02	5,820	5,311	4.83	4.83	805	702	8.00	1.88	777	649	10.05	2.23
	37	60	6.57	2.41	5,343	4,688	6.68	6.68	786	653	10.83	3.13	732	630	10.92	3.21

TABLE 13.—*Final summary of average values for the Wisconsin and Arkansas jobs*

Item	Wisconsin State-aid project 2926	Wisconsin State-aid project 2916	Federal-aid project 259–A, Jefferson, Co., Ark.
Number of batches sampled	118	80	44
Proportions by weight, per cent:			
Coarse aggregate	52.56	52.11	48.56
Sand	30.11	30.34	33.49
Cement	11.35	11.28	11.78
Water	5.97	5.73	6.01
Variation factor within batch, per cent	3.90	4.37	4.75
Cement content, sacks per cubic yard	5.16	5.18	5.19
Water content, gallons per sack of cement	5.96	5.81	5.82
Ratios by absolute volume:			
a/c (sand to cement)	3.02	3.03	3.38
b/a (gravel to sand)	1.72	1.72	1.45
b/b_0 (see p. 275)	.746	.750	.751
Mix by weight:			
Sand	2.67	2.71	2.86
Coarse aggregate	4.67	4.68	4.18
AVERAGE STRENGTH DATA			
Compressive strength of cylinders at 28 days, pounds per square inch	3,646	3,672	5,039
Variation, per cent	6.33	5.66	5.89
Flexural strength of beams in pounds per square inch:			
At 7 days	598	595	676
Variation, per cent	4.55	4.70	6.56
End break at 28 days	872	877	723
Variation, per cent	4.03	5.22	6.04
Center break at 28 days	866	868	698
Variation, per cent	4.76	4.95	5.48
SIEVE ANALYSIS			
Percentage retained on—			
2½-inch	4	3.6	
1½-inch	14.8	13.6	21.1
¾-inch	44.3	38.1	56.3
¼-inch	36.9	44.6	22.6
MAXIMUM AND MINIMUM VALUES[1]			
Maximum cement content, sacks per cubic yard	5.49	5.64	5.72
Minimum cement content, sacks per cubic yard	4.83	4.69	4.63
Maximum water content, gallons per sack of cement	6.38	6.55	6.69
Minimum water content, gallons per sack of cement	5.58	5.15	5.12
Strength of cylinders at 28 days in pounds per square inch:			
Maximum	4,031	4,012	5,323
Minimum	3,266	3,325	4,756
7-day beam strength in pounds per square inch:			
Maximum	633	631	737
Minimum	562	560	621
28-day beam strength in pounds per square inch:			
End-break maximum	919	934	779
End-break minimum	826	819	673
Center-break maximum	919	918	748
Center-break minimum	814	810	648

[1] Averages of maximum and minimum values obtained on each batch.

or to differences in the performance of the old and new pavers. It is apparent from an inspection of this table that no definite relation can be established between size of batch and batch uniformity or uniformity of breaking strength. It is also evident that the uniformity values given by the old mixer are of the same order as those given by the new.

STOP-WATCH STUDIES ANALYZED

The data regarding the distribution of the materials obtained on these jobs by means of the separation or wash analyses and the data on breaking strength of both beams and cylinders indicate rather clearly that, when two or more sizes of coarse aggregates are used, the modern 27E paver, in good condition, will mix a 35-cubic-foot batch to as high a degree of uniformity as it will a 27-cubic-foot batch, and that the strength of the concrete from the larger batch will be equally as good as that from the smaller. This does not prove that the larger batch will always be the most economical. While theoretical considerations, as well as the unit prices of the successful bidders on the two Wisconsin projects, indicate a certain advantage for the larger batch, these are not conclusive, as is shown by the fact that several bidders bid the same price for all of the proposed batch sizes. Very careful and detailed production studies were made on these jobs in order to obtain some actual data as to what effect the use of the larger batches might have on the rate of production. These included daily stop-watch studies of the mixer for 1 hour in the morning and 1 hour in the afternoon

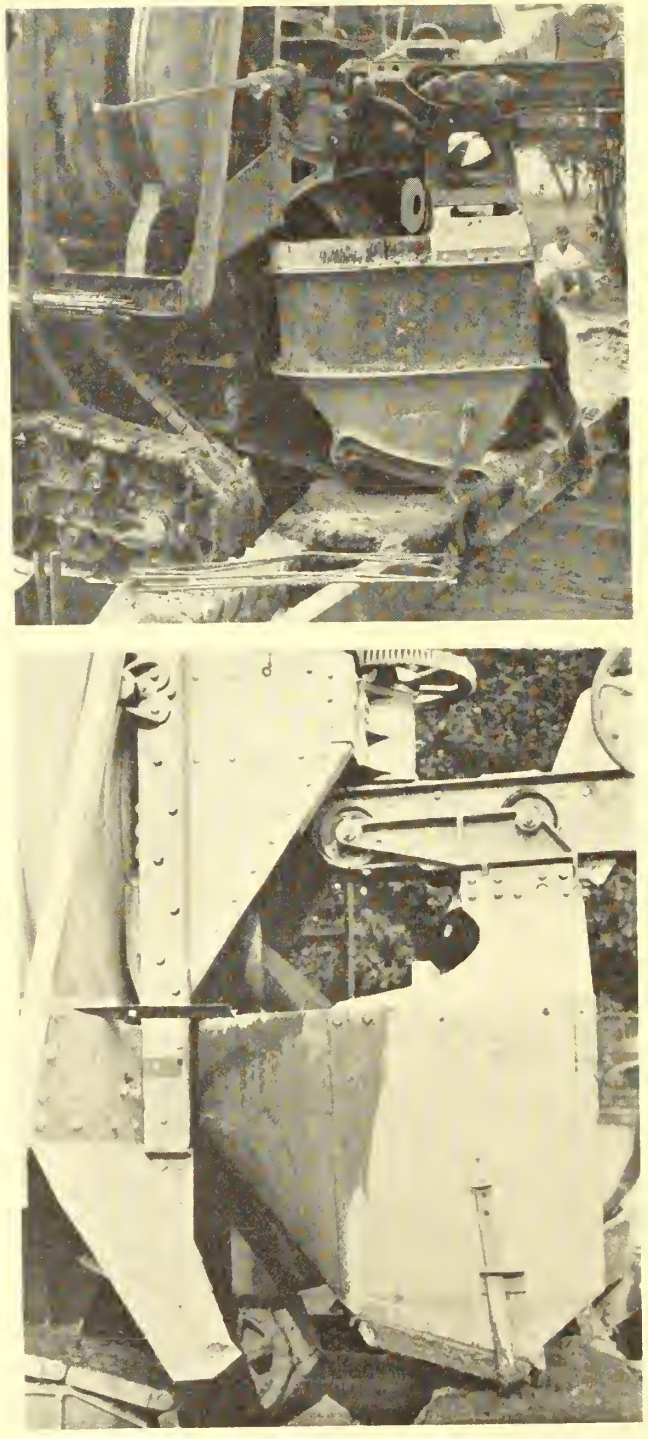


FIGURE 6.—UPPER PHOTOGRAPH SHOWS TYPICAL SPREADER-BUCKET DESIGN USED ON OLD PAVERS. BELOW IS SHOWN NEW DESIGN OF SPREADER BUCKET FOR 1930 AND 1931 PAVERS

TABLE 14.—*Degree of nonuniformity in mixing and variation in breaking strength of standard cylinders, as shown by data obtained on Wisconsin State-aid project 2926. Average values are given for five samples and five cylinders from each batch*

OLD MIXER

[Size of batch in cubic feet]

27		30		33		35	
Batch variation	Cylinder variation	Batch variation	Cylinder variation	Batch variation	Cylinder variation	Batch variation	Cylinder variation
Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
2.91	5.34	3.81	4.59	5.20	4.65	6.48	7.63
5.40	6.11	5.79	8.38	5.10	4.79	3.32	4.32
2.17	4.15	4.98	5.21	6.12	8.38	8.41	2.65
4.00	3.50	9.33	11.27	6.52	5.29	5.49	5.23
5.61	8.06	4.61	3.62	5.68	5.08	3.32	6.69
2.78	3.25	4.33	9.16	1.43	3.83	4.99	3.36
2.92	5.89	7.72	4.77	4.53	6.47	3.49	7.92
3.17	1.51	5.47	5.97	5.83	8.22	3.83	6.72
6.60	5.11	2.75	11.80	4.25	4.28	2.03	13.63
Av. 3.95	4.77	5.42	7.20	4.96	5.66	4.59	6.47

NEW MIXER

4.13	5.67	4.38	6.57	3.86	4.04	3.80	6.25
3.69	5.91	4.13	4.65	2.54	3.55	5.65	6.19
2.86	5.90	4.30	4.15	2.45	4.59	3.21	4.86
4.56	2.75	2.89	4.70	2.09	4.96	3.02	7.60
4.73	6.70	4.01	13.41	5.11	10.22	2.51	2.64
2.74	14.51	3.67	4.77	4.49	5.27	3.28	4.20
2.03	10.43	2.90	5.05	4.62	5.35	5.21	4.06
2.63	3.43	4.48	6.80	4.31	7.98	3.08	6.36
2.79	7.03	6.65	10.59	4.11	4.20	2.09	6.95
4.59	12.50	2.38	6.78	2.21	7.90	5.54	9.34
Av. 3.48	7.48	3.98	6.75	3.58	5.81	3.74	5.84

to determine both the average mixer cycle and the duration and cause of all time losses or interruptions to continuous mixer operation. During the stop-watch studies the time in seconds required to raise the skip, the time to mix, and the time from the bell to the instant the skip began to rise for another cycle, were determined. If an interruption occurred at any point in the mixer cycle, the duration and the cause of each stop or delay were recorded. If the delay or stop was 15 minutes or more in duration it was classified as a major delay, and if it was less than 15 minutes it was classified as a minor delay.

Auxiliary stop-watch studies were also made of the several component parts of the mixer-operating cycle and included the following: Time to raise the skip; the instant the water and solid materials began to enter the mixer drum, the time required to discharge the water and solid materials into the drum; the time from the instant the skip reached a vertical position until the batchmeter bell rang; the time from when the bell rang until the instant the concrete appeared in the discharge chute; the time to discharge the concrete; the time from the bell until the skip again started up; and finally, the revolutions of the mixer drum. The mixer drum was also carefully watched for overloading, spillage, leakage, clogging, possible retarding of the mixer drum speed, and the appearance of the batch in the mixer drum (whether it appeared to be uniformly mixed, etc.). A careful check was made each day of the contractor's personnel, unit costs, amount of concrete placed, yield, etc.

EFFICIENCY OF PLANT AND EQUIPMENT STUDIED

In order to fix definitely the casues of such delays as occurred or were anticipated, the auxiliary equipment, such as the cranes, batcher bins, and finishing machines, were studied to determine their relation to the rate of

FIGURE 7.—PLANT SET-UP USED ON STATE-AID PROJECT 2916, SHEBOYGAN COUNTY, WIS., ON WHICH THREE SIZES OF COARSE AGGREGATE AND ONE OF SAND WERE USED. MATERIALS USED WERE DELIVERED BY RAIL

production. Views of the plant layouts on the two Wisconsin jobs are shown in Figures 7, 8, 9, and 10. Figure 11 shows the type of 1927 Model 27E paver used on the Wisconsin jobs. The contractors on these two jobs each had old pavers of this type when the projects were started. As soon as the different-sized batches had been studied in regard to quality tests and production on the old pavers, new 1930 Model 27E pavers were loaned by the manufacturers for similar studies.

FIGURE 8.—TYPICAL PLANT SET-UP USED ON STATE-AID PROJECT 2926, SHEBOYGAN COUNTY, WIS. MATERIALS HAULED IN BY TRUCK FROM A LOCAL PIT. COARSE AGGREGATE SEPARATED IN THREE SIZES

PRODUCTION RATE OF NEW PAVERS SHOWS SUBSTANTIAL INCREASE OVER OLD TYPE

On the old mixers a 2½-inch strip of metal was placed around the back part of the spreader-buckets to increase their carrying capacity. Even with this extra metal strip it was found that the 35 cubic-foot batches could not be discharged in one operation. As a consequence, double discharge had to be used for the 35-cubic-foot batches, which proved to be uneconomical. However, if the spreader buckets had been slightly larger a saving could have been made by using the larger 35-cubic-foot batch. No alterations had to be made on the

spreader buckets of the new pavers in order to carry the 35-cubic-foot batches, which the new pavers could handle with ease.

The mixer drums of both the old and new pavers mixed up to 35-cubic-foot batches without spillage, leakage, or clogging. The computed values given in Table 15 were made from measurements of the mixer drum to illustrate the theoretical carrying capacity of the old and the new pavers when used on level grade and on a 6 per cent grade.

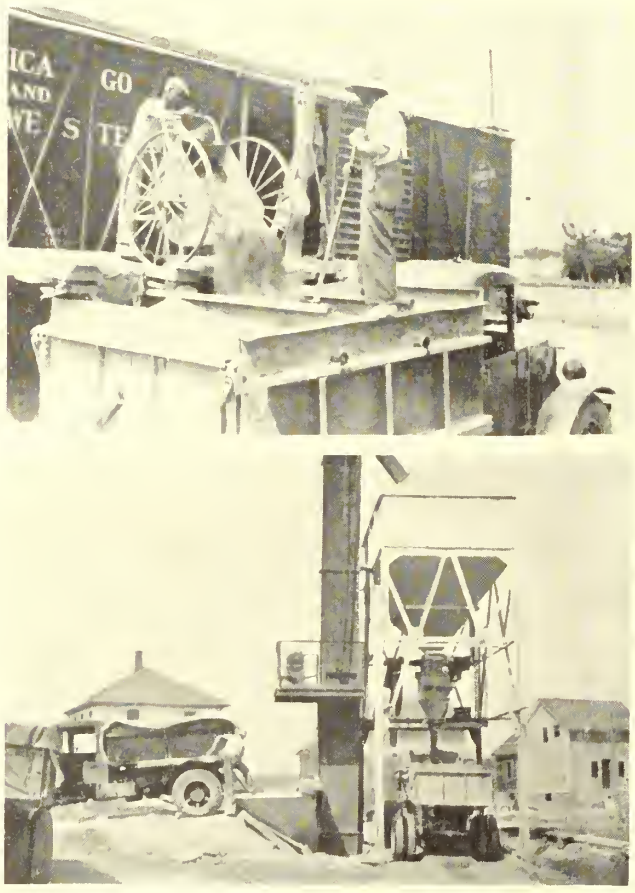


FIGURE 9.—METHODS OF HANDLING BULK CEMENT ON WISCONSIN STATE-AID PROJECTS 2916 AND 2926

TABLE 15.—*Computed capacity, in cubic feet, of mixer drums of old and new 27E Pavers*

	Total volume content	Water level content, level grade	Capacity of buckets and blades, level grade	Working content on level grade	Water level content, 6 per cent, grade	Capacity of buckets and blades, 6 per cent grade	Working content on 6 per cent grade
Old paver: 6 blades, 6 buckets, boom bucket, capacity, 36¾ cubic feet	94.75	25.84	11.00	36.84	22.00	11.00	33.20
New paver: 5 blades, 10 buckets, boom bucket, capacity, 42 cubic feet	100.18	28.50	10.20	38.50	24.60	10.20	34.80

The larger batch sizes increased the mixer cycle slightly because of the longer time required to discharge the batch from the drum and the longer time required for the materials to discharge from the skip into the mixer drum. Tables 16 and 17 show the actual average mixer cycles as determined by the stop-watch studies for a batchmeter setting of 60 seconds, as most generally used on the two Wisconsin jobs, together with the effect of the various batch sizes on the maximum possible rate of production. Tables 18 and 19 show the same data on the basis of actually mixing all the solid materials for either 50 or 60 seconds. It should be noted that the mixing time of the concrete does not terminate when the batchmeter rings but continues

FIGURE 10.—LOCAL PIT PLANT LAYOUT EQUIPPED FOR PRODUCING THREE SIZES OF COARSE AGGREGATE FOR WISCONSIN STATE-AID PROJECT 2926

until the concrete appears in the discharge chute. This time interval is usually termed the discharge lag.

Allowance should be made in the batchmeter setting to take care of this lag. In setting the batchmeter the following formula should be used:

$$A+B-C=D$$

where A is the mixing time, B the lag of the solid materials after the skip reaches the vertical C the discharge lag from the bell to the appearance of the concrete in the discharge chute, and D the batchmeter setting in seconds. The water added at the paver should enter

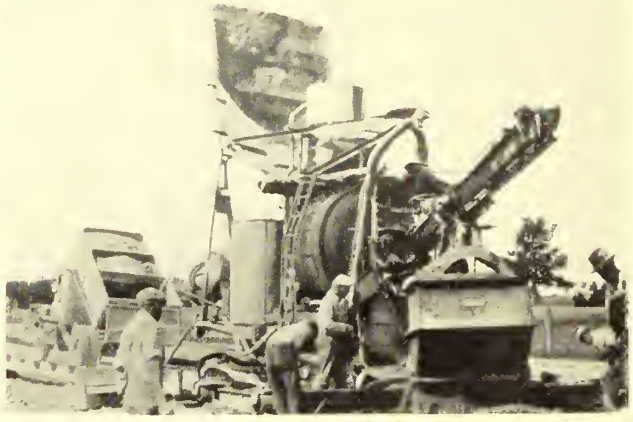


FIGURE 11.—TYPE OF 1926 MODEL 27E PAVER USED ON WISCONSIN STATE-AID PROJECTS 2916 AND 2926

the paver drum about 1½ seconds before the solid materials (aggregate and cement) and lag after the solid materials no more than 10 seconds. This combination of lags will result in more efficient charging of the materials and will keep the throat of the mixer clear, as well as the blades and buckets.

There is also a mixing action in progress on a considerable portion of the batch while the drum is being both charged and discharged. This, however, is entirely omitted in determining the length or duration of the mixing time.

The difference in the mixer cycles between the old paver and the new paver was found to be surprisingly

TABLE 16.—*Analysis of production of 3-year old 27E paver as affected by batch size, with the batchmeter setting at 60 seconds*

Batch size (cubic feet)	27	30	33
Mixer cycle, seconds	74.90	76	77.14
Possible batches per hour	48.06	47.37	46.67
Percentage decrease in batches per hour due to batch size		1.43	2.89
Possible cubic yards per hour	48.06	52.63	57.04
Percentage net increase in possible production due to batch size		9.5	18.7

TABLE 17.—*Analysis of production of new 27E paver as affected by batch size, with the batchmeter setting at 60 seconds*

Batch size (cubic feet)	27	30	33	35
Mixer cycle, seconds	68.60	68.60	69.16	69.77
Possible batches per hour	52.48	52.48	52.05	51.60
Percentage decrease in batches per hour due to batch size			.82	1.70
Possible cubic yards per hour	52.48	58.31	63.62	66.87
Percentage net increase in possible production due to batch size		11.1	21.2	27.4

TABLE 18.—*Analysis of production of 3-year old 27E paver as affected by batch size and mixing time*

Batch size (cubic feet)	27	30	33	27	30	33
Mixing time of all solid materials, seconds	60	60	60	50	50	50
Mixer cycle, seconds	76	77.82	79.69	66	67.82	69.69
Possible batches per hour	47.37	46.26	45.18	54.54	53.08	51.66
Percentage decrease in possible batches per hour due to batch size, based on a 27-cubic-foot batch		2.34	4.66		2.68	5.28
Possible cubic yards per hour	47.37	51.40	55.22	54.54	59	63.15
Percentage net increase in possible production due to batch size, based on a 27-cubic-foot batch mixed 60 seconds		8.5	16.6	15.2	24.6	33.3

TABLE 19.—*Analysis of production of new 27E paver as affected by batch size and mixing time*

Batch size (cubic feet)	27	30	33	27	30	33
Mixing time of all solid materials, (seconds)	60	60	60	50	50	50
Mixer cycle (seconds)	69.45	70.69	71.89	59.45	60.69	61.89
Possible batches per hour	51.81	50.93	50.08	60.55	59.32	58.17
Percentage decrease in possible batches per hour due to batch size, based on a 27-cubic-foot batch		1.75	3.40		2.02	3.92
Possible cubic yards per hour	51.84	56.59	61.21	60.55	65.93	71.09
Percentage net increase in possible production due to batch size and mixing time, based on a 27-cubic-foot batch mixed 60 seconds		9.2	18.1	16.8	27.2	37.2

large. Because of this shorter mixing cycle alone, it is possible to obtain an increase of 10 per cent or more in production by using the new pavers. This fact is brought out by the data given in Table 20.

Because of the shorter mixing cycle and the better mechanical condition of the new paver, its actual advantage in effecting increased production was even greater than indicated above. The actual average hourly rates of production on Wisconsin State-aid project 2926 from the old and the new pavers for each batch size during the period of the studies are given in Table 21.

AUXILIARY EQUIPMENT KEPT PACE WITH PAVERS

During the time when the 50-second mixing time and the 33-cubic-foot batches were used, the finishing machine and finishing operations could, and did, readily keep up. This is true as well for the other auxiliary operations. This combination of mixing time and batch size did not therefore require more equipment or larger equipment. The rate of production

TABLE 20.—*Comparison of rates of production of old and new 27E pavers*

Size of batch (cubic feet)	27	30	33	27	30	33
Mixing time of all solid materials (seconds)	60	60	60	50	50	50
Possible production per hour for new paver (square feet)	119.83	130.81	141.49	110	152.44	161.37
Possible production per hour for old paver (square feet)	109.50	118.82	127.65	126.10	136.39	145.97
Percentage increase in possible production with new paver	9.4	10.1	10.8	11.0	11.8	12.6

TABLE 21.—*Average hourly rates of production on Wisconsin job 2916 obtained with old and new pavers, for a 60-second setting of batchmeter*

Size of batch	Square yards per hour		Percentage increase over old paver using 27-cubic foot batch
	Old paver	New paver	
27 cubic feet	188.1	217.2	15.5
30 cubic feet	218.5	257.6	36.9
33 cubic feet	222.5	260	37.3
35 cubic feet		271	44.1

was still well within the capacity of the other coordinating units with no particular increase in labor or equipment. It seems likely, however, that a greater output than that now possible with the 33-cubic-foot batch mixed 50 seconds will, under present methods of operation, require additional or larger equipment.

On the basis of this investigation the State of Wisconsin is now permitting the use of a 33-cubic-foot batch and a 50-second mixing time for all the solid materials. The 33-cubic-foot batch allowance in Wisconsin is based on the fact that the spreader bucket of practically all 27E pavers now in use will hold this size batch in one discharge. The two old pavers and the two new pavers as used on these jobs handled the 33-cubic-foot batch equally as well as they did the smaller-sized batches.

CHART SHOWS METHOD OF DESIGNING MIXES

The mixes on both jobs were designed for the minimum allowable cement content and maximum allowable water content. The workability of the concrete was maintained through the adjustment of the gravel and sand content by means of the workability factor, b/b_o, as developed by Talbot and Richart.[2] Figure 12 is a chart designed for the purpose of obtaining the proportions of sand and coarse aggregate required to produce concrete having the desired workability factor. The chart is based on a cement content of 5 sacks per cubic yard of concrete and 6 gallons of water per sack of cement, which were the specifications for the Wisconsin jobs.

The process of determining the proportions of the mix is illustrated by the dash lines on the chart, which give the solution for a workability factor of 0.76, a value which seemed to work best on both of these jobs.

The percentage of voids in the course aggregate (dry loose) as used for the example in Figure 12 is 36. Enter the chart at the bottom where the figures for the voids in the coarse aggregate are given. Proceed from the number 36 and follow the dash line upward to the ordinate (point 1) where it intersects the workability ratio, $b/b_o = 0.76$. The absolute volume of

[2] Bulletin No. 137, Engineering Experiment Station, University of Illinois, 1923.

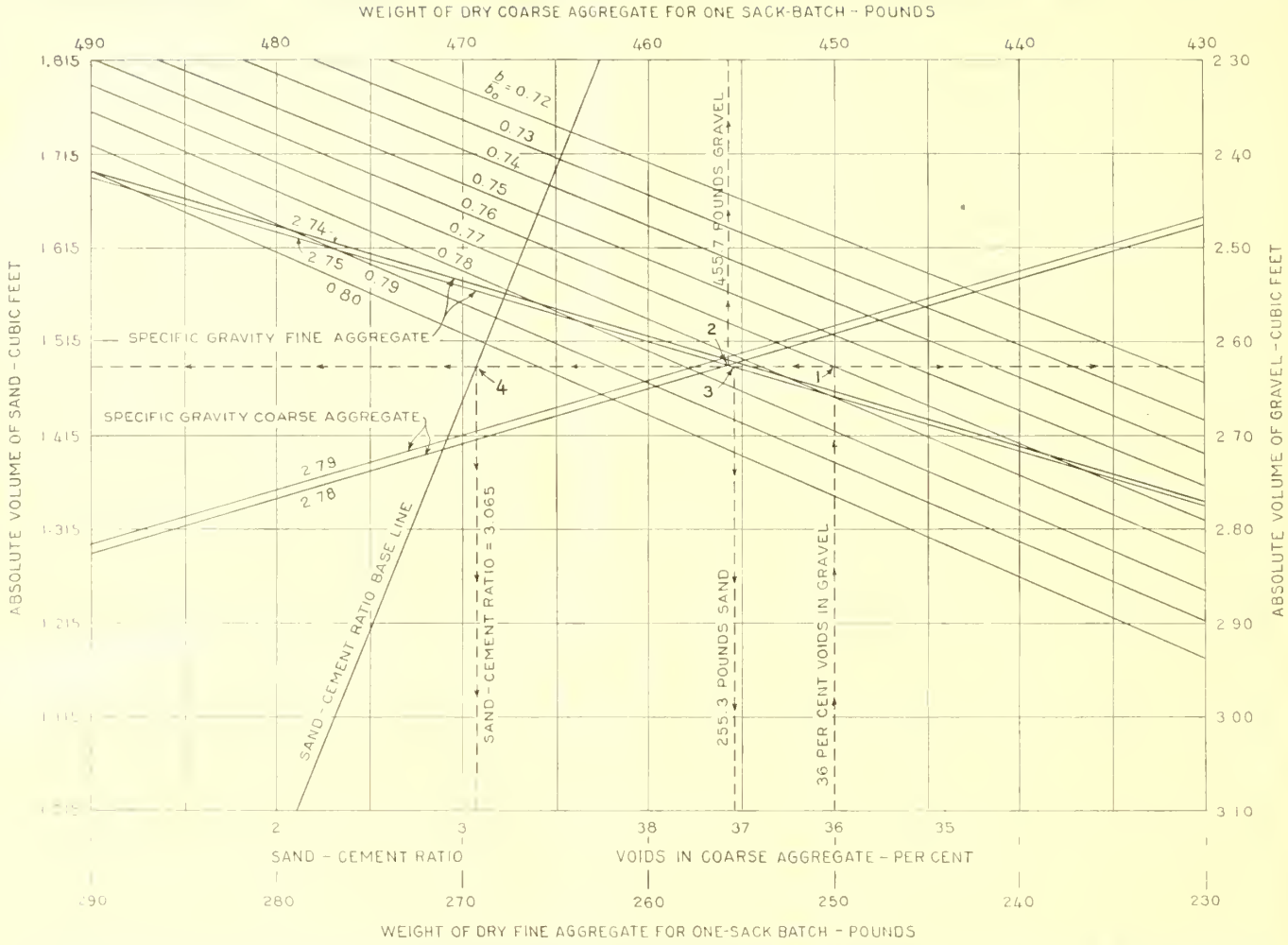


FIGURE 12.—CHART FOR OBTAINING REQUIRED PROPORTIONS OF BATCH FOR A GIVEN VALUE OF b/b_0, ON WISCONSIN STATE-AID PROJECTS 2916 AND 2926. CHART IS BASED ON A CEMENT CONTENT OF 5 SACKS OF CEMENT PER CUBIC YARD OF CONCRETE AND 6 GALLONS OF WATER PER SACK OF CEMENT

coarse aggregate is given by the same ordinate on the right-hand scale. Follow the dashline to the left to the specific gravity line of 2.78 for the coarse aggregate (point 2) and upward, and read the weight of dry coarse aggregate for a 1-sack batch, 455.7 pounds, on the upper scale. Follow the dash line from the percentage of voids in the coarse aggregate (36) again to the value $b/b_0 = 0.76$, then to the left to the specific gravity of 2.75 for the fine aggregate (point 3), and then down, to read the weight of 255.3 pounds of sand for a 1-sack batch on the lower scale at the bottom of the chart. Proceed again from the intersection of the 36 per cent voids line with the value $b/b_0 = 0.76$ and follow the dash line to the left to the sand-cement ratio base line (point 4), and then downward, to read the sand-cement ratio by absolute volume, 3.065, on the upper left-hand scale at the bottom of the chart. The absolute volume of the sand is read on the left-hand scale.

PROPORTIONING AND SIEVE ANALYSIS OF AGGREGATES DISCUSSED

The three sizes of aggregates were also recombined experimentally, different percentages of each size being used, in order to obtain as low a void content as practicable without sacrifice of gradation in obtaining a workable concrete.

The proportions of the various sizes of coarse aggregate for Wisconsin State-aid project 2926, as required by the specifications, and the sieve analyses of these aggregates are given in Table 22. Sieve analyses of the sand used on both Wisconsin jobs are given in Table 23.

PLANT LAYOUTS SHOWN

Figure 13 shows the plant layout and set-up for using three sizes of coarse aggregate on Wisconsin State-aid project 2916. It is obvious that a very crowded condition existed. In fact, the plant set-up was such that trucks lost on an average of 2 minutes per load more than necessary for a typical set-up, a condition which required the use of one and sometimes two trucks more than would have been used normally. An extra crane was also required. The materials were delivered by rail.

Figure 14 is the same plant site but with equipment and stock-piles planned for handling two sizes of coarse aggregate. In so far as the cost of handling the aggregates and batching is concerned, this set-up would not result in any more cost for handling the two sizes than if only one size of coarse aggregate were used.

Figure 15 is the plant layout for one of the set-ups on Wisconsin State-aid project 2926, showing the number of turns and maneuvers that the trucks had to make in order to obtain a load. Figure 16 shows the same set-up but with a recommended change that could be used on similar jobs. Turning time for the

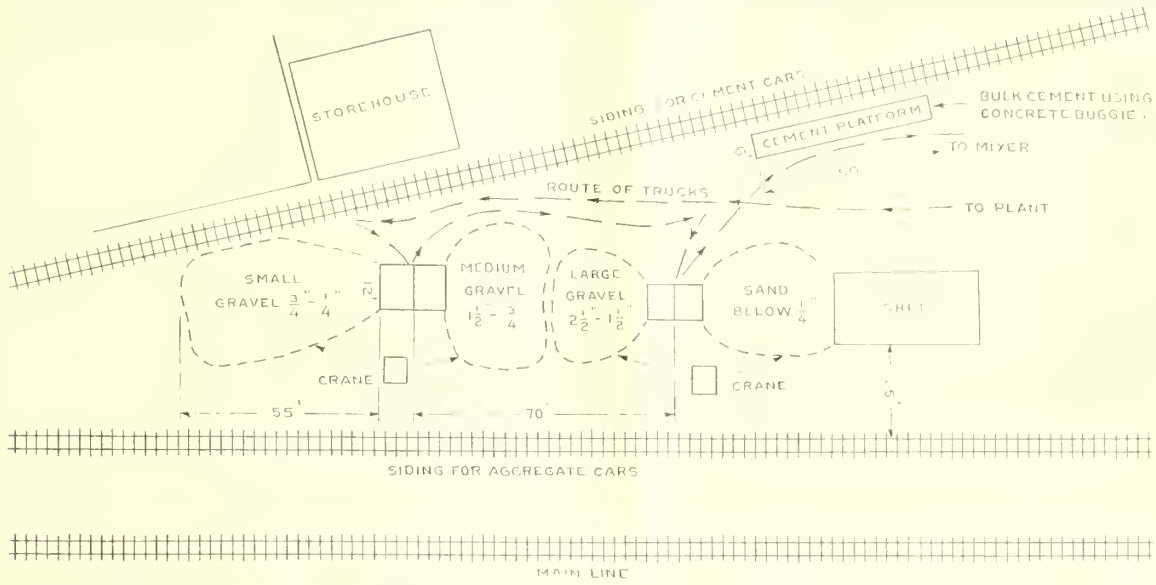


FIGURE 13.—PLANT LAYOUT FOR WISCONSIN STATE-AID PROJECT 2916, THREE SIZES OF COARSE AGGREGATE

TABLE 22.—*Proportions and sieve analyses of coarse aggregates used on Wisconsin State-aid project 2926*

PROPORTIONS

No.	Size	Proportions
		Per cent
1	2½ to 1½ inches	30–35
2	1½ to ¾ inch	45–50
3	¾ to ¼ inch	15–25

SIEVE ANALYSES

	Per cent
No. 1 coarse aggregate—	
Retained on 3-inch circular opening	0
Retained on 2½-inch circular opening	0–10
Retained on 1½-inch circular opening	90–100
Retained on 1¼-inch circular opening	100
No. 2 coarse aggregate—	
Retained on 1¾-inch circular opening	0
Retained on 1½-inch circular opening	0–10
Retained on ¾-inch circular opening	90–100
Retained on ½-inch circular opening	100
No. 3 coarse aggregate—	
Retained on 1-inch circular opening	0
Retained on ¾-inch circular opening	0
Retained on ¼-inch circular opening	95–100
Retained on No. 10 sieve	100

TABLE 23.—*Sieve analyses of sands used on Wisconsin State-aid projects 2916 and 2926*

	Per cent
Project 2916:	
Retained on No. 4 sieve	6.86
Retained on No. 20 sieve	48.28
Retained on No. 50 sieve	79.79
Retained on No. 100 sieve	93.39
Project 2926:	
Retained on No. 4 sieve	1.56
Retained on No. 20 sieve	39.28
Retained on No. 50 sieve	83.82
Retained on No. 100 sieve	95.12

trucks would then be eliminated. If only two sizes of coarse aggregate had been specified, a 3-compartment bin could have been used which would have required but one stop for the trucks. This would have been a most economical set-up and would still have utilized the advantage of multiple-sized aggregates. On this job the material was delivered by truck from a local pit and only one crane was necessary to supply the mixer demand for aggregates. This crane supplied material for the mixer when a 33-cubic-foot batch and a 50-second mixing time were used. An average of fifty-five 33-cubic-foot batches an hour were taken care of by the crane. This means that the crane handled an average of 238,931 pounds of sand and gravel, or about 80 cubic yards of material, per hour.

Figure 17 shows a set-up observed during the summer of 1930 handled by one crane and utilizing a method of straight-line loading. Bulk cement was placed in the batch between the stone and sand, eliminating the time usually required for covering the cement. This method proved very satisfactory. Some engineers, however, prefer to have the order of loading changed to second-size aggregate, sand, cement, and then coarse aggregate.

Incidentally, this contractor was using sack cement but obtained permission to try bulk cement, which resulted in a saving of 11 cents on each barrel of cement. This item alone was worth while. This procedure also assured the proper weight of cement each time, enabled the contractor to maintain a constant maximum batch, and eliminated the cost of handling the empty sacks. The cement retained in the empty sacks averaged about 0.4 pound per sack. A State buying cement can therefore make a large saving by eliminating this item. As the advantage of bulk cement becomes more generally appreciated it is believed that its use will be extended. Up to the present time the most popular way of handling bulk cement at the batcher plant, and the way used in this case, is by the use of 2-wheeled concrete buggies. Two cars of cement are unloaded at the same time. A wooden platform is constructed to the same elevation as the box-car floor with a length sufficient to reach the doors of both cars. Usually two men are used in each car for loading the buggies, two men to wheel the buggies to a platform scale, two men to weigh and finally dump the cement into the trucks.

On Wisconsin job 2916, as noted in the sketches, the bulk cement was delivered by rail and concrete buggies were employed to handle the cement. On Wisconsin job 2926 the cement was delivered to the batching plant in trucks and the cement loaded mechanically. Both methods proved very satisfactory and the trucks could be loaded rapidly with no apparent loss of cement during loading or in transit. The sand

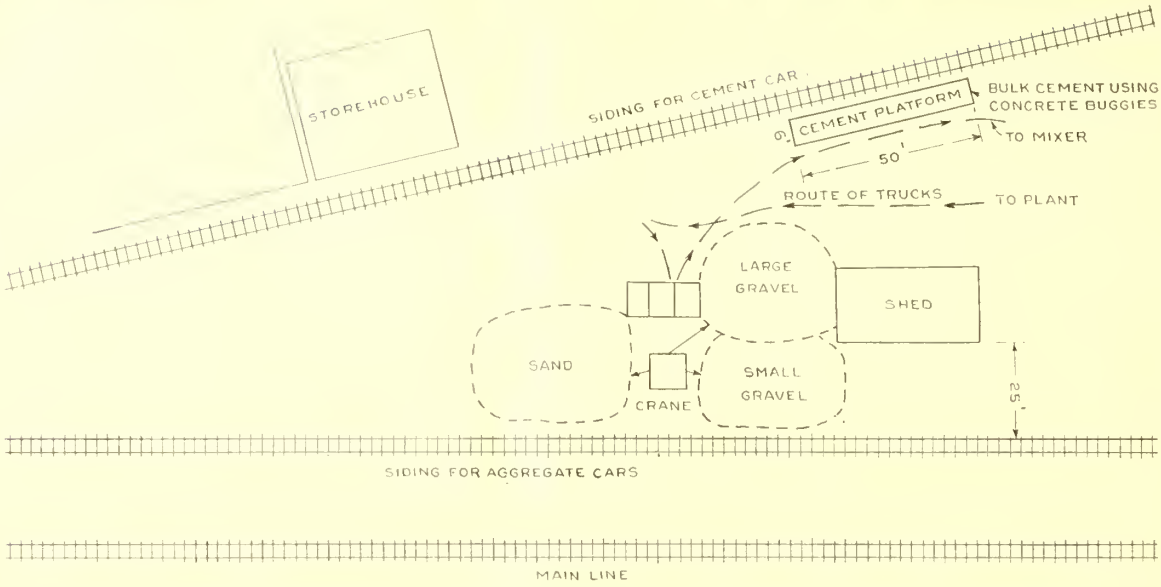


FIGURE 14.—SUGGESTED IMPROVEMENT IN PLANT LAYOUT FOR WISCONSIN STATE-AID PROJECT 2916, TWO SIZES OF COARSE AGGREGATE

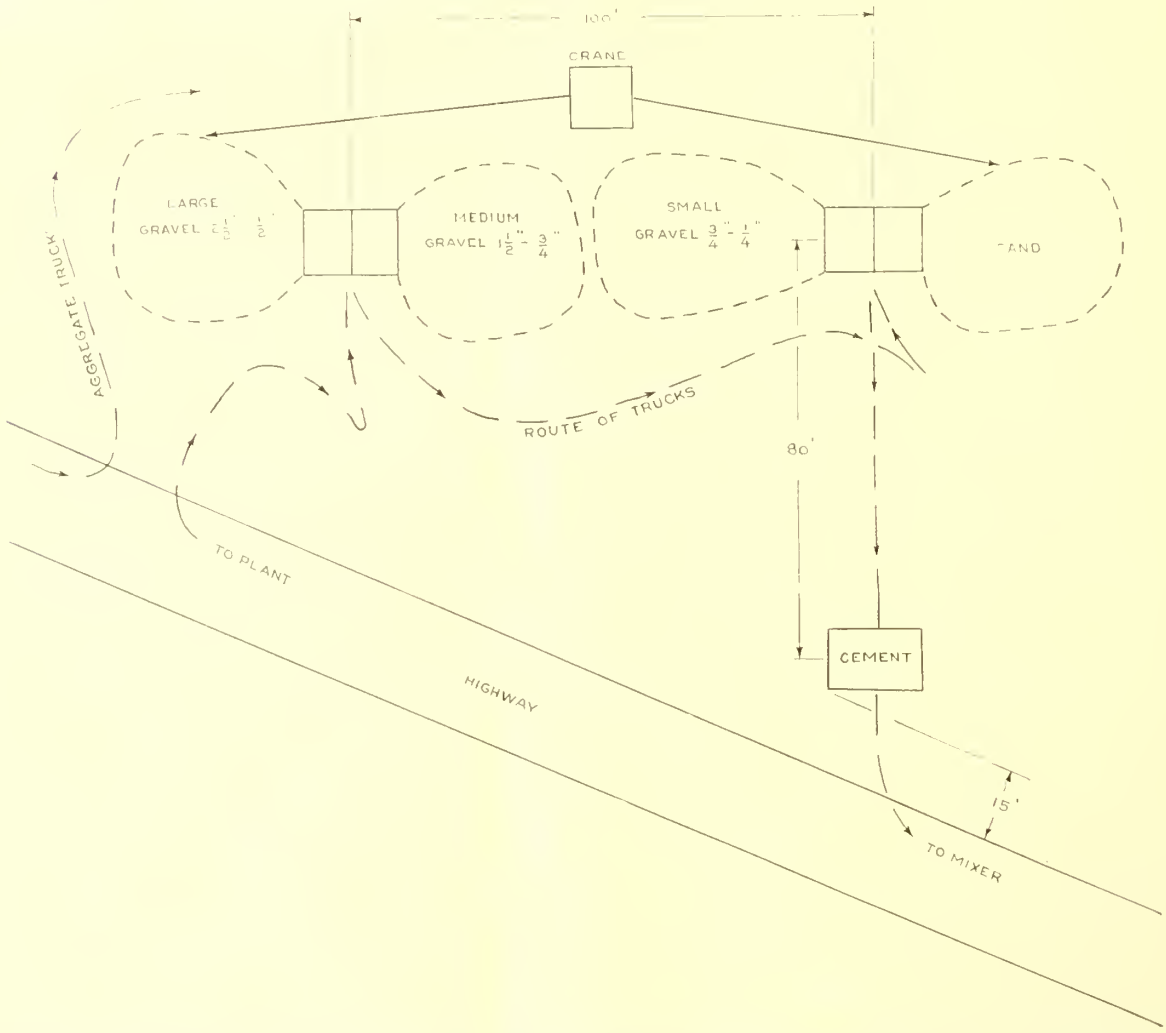


FIGURE 15.—PLANT LAYOUT FOR WISCONSIN STATE-AID PROJECT 2926, THREE SIZES OF COARSE AGGREGATE

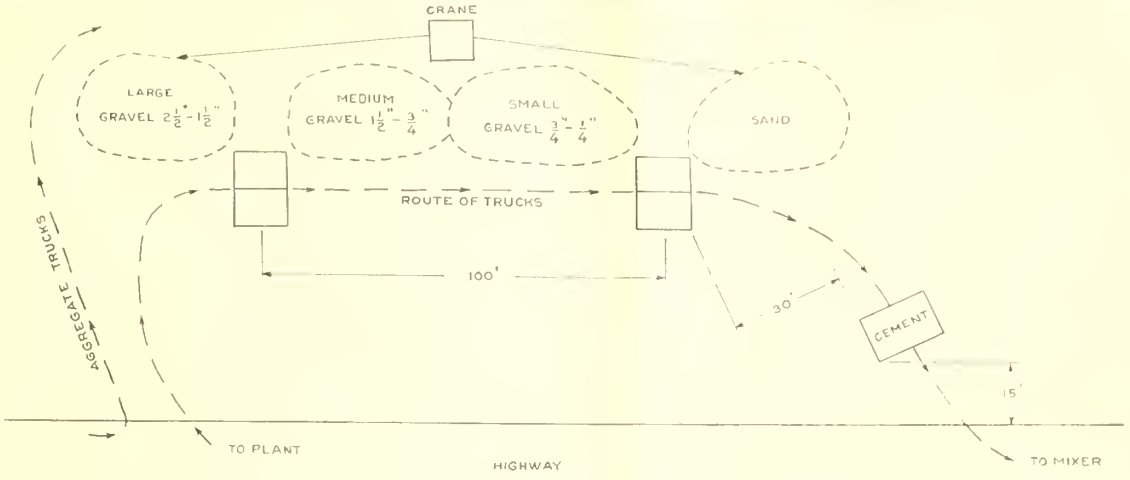


FIGURE 16.—SUGGESTED IMPROVEMENT IN PLANT LAYOUT FOR WISCONSIN STATE-AID PROJECT 2926, THREE SIZES OF COARSE AGGREGATE

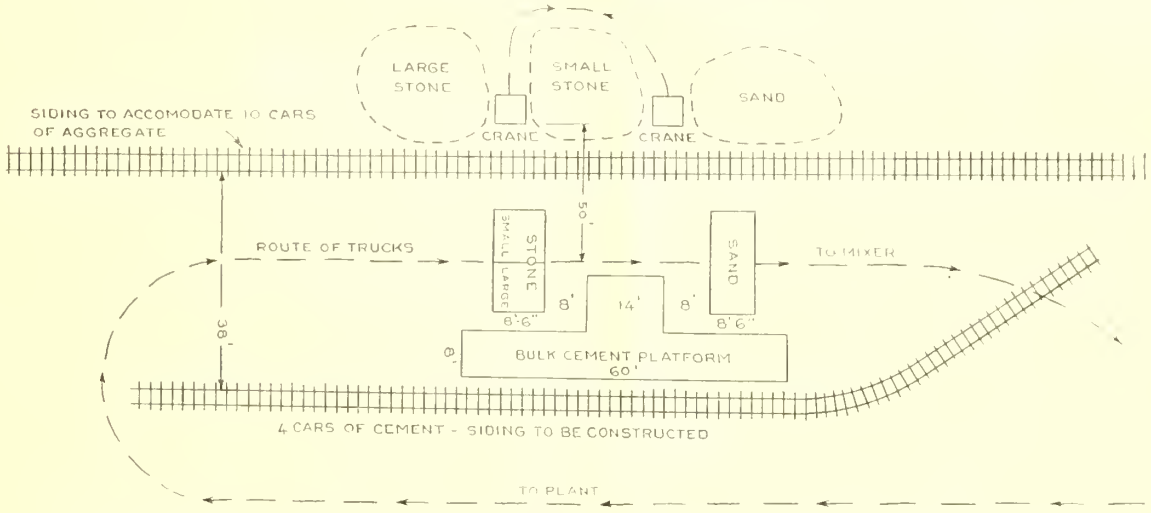


FIGURE 17.—TYPICAL PLANT LAYOUT FOR STRAIGHT-LINE LOADING, ADAPTED TO BULK CEMENT. TRUCKS LOADED MECHANICALLY OR WITH CONCRETE BUGGIES; SAND USED FOR CEMENT COVERAGE

was dumped on the coarse aggregate so that it could be pocketed and the bulk cement covered with the loose sand.

Under no condition should the bulk cement be dumped at the rear of the truck body, as this retards the flow of material into the skip, clogs the skip throat and mixer throat, and causes the blades and buckets to be badly coated, which results in delaying the mixer operations.

Both jobs employed a straight-line loading of the trucks for the bulk cement, which reduced the amount of turning and backing required in taking on a load. Delaying a 3-batch truck 1 minute represents a loss of about 5 cents. This item may seem small but it accumulates rapidly into a large sum. Delaying the mixer 1 minute often represents the loss of about $1. On a paving job where operations should synchronize and coordinate, minutes lost are dollars wasted. An efficient batching plant removes many chances for delay.

STOP-WATCH ANALYSIS SHOWS FEW DELAYS CAUSED BY USE OF SEPARATE-SIZED AGGREGATES

Tables 24 and 25 are summaries from the time studies which were made during the period from June to October on the two Wisconsin jobs. These data were obtained by means of careful, daily stop-watch studies of all operations connected with the mixer. Delays less than 15 minutes in duration were classified as minor delays, and delays greater than 15 minute in duration were grouped as major delays.

These data indicate clearly that delays caused by the use of separate sizes of coarse aggregate were negligible, except on the first Wisconsin job where the plant space was so limited that the use of four stock-piles produced such crowded conditions that it was very difficult to maintain efficient truck operation. The job started with one crane but as production was improved another crane had to be put on the job. Probable causes of other delays were also indicated from time to time by the stop-watch studies, but the ready cooperation and alertness of the contractors in acting on these suggestions obviated all delays of any consequence due to handling the separated aggregates even under these rather unfavorable conditions.

CHARTS SHOW DISTRIBUTION OF COSTS

Figures 18 to 23 show some of the pertinent data collected on the two Wisconsin jobs. These charts illustrate the costs involved in constructing the concrete pavement. These two jobs had practically the

TABLE 24.—*Summary of time losses and their effect on production, for Wisconsin State-aid project 2916, June 23 to August 21, 1930*

MAJOR DELAYS OCCURRING DURING AVAILABLE WORKING TIME

Character of delays	Hours	Per cent
Hauling supply	0.25	0.05
Mixer trouble, mechanical	4.08	.75
Water supply	3.63	.67
Lack of materials at yard	.77	.14
Crane trouble, mechanical	10.27	1.88
Moving	83.45	15.30
Rain	13.80	2.53
Wet subgrade	21.25	3.90
Miscellaneous	1.61	.30
Total	139.11	25.52
Time major equipment actually operated	406.07	74.48
Available working time	545.18	100.00

MINOR DELAYS OCCURRING DURING TIME OF ACTUAL OPERATION

Character of delays	Hours	Per cent
Hauling supply	14.95	3.68
Hauling operation	11.09	2.73
Dumping	2.62	.64
Mixer trouble, mechanical	3.66	.90
Mixer operation	9.47	2.33
Water supply	12.88	3.17
Lack of materials at yard	.33	.08
Set parting strip	.51	.13
Subgrade not prepared	6.98	1.72
Place reinforcing steel	1.34	.33
Wait for finishers	2.04	.50
Sand batch, lip curb	1.49	.37
Expansion joint	.38	.09
Crane operation	3.03	.75
Miscellaneous	2.88	.71
Total	73.65	18.14
Time major equipment operated at 100 per cent efficiency	332.42	81.86
	406.07	100.00

Actual production, 15,945 batches, 93,395 square yards; over-all efficiency of major equipment operation, 83.1 per cent.

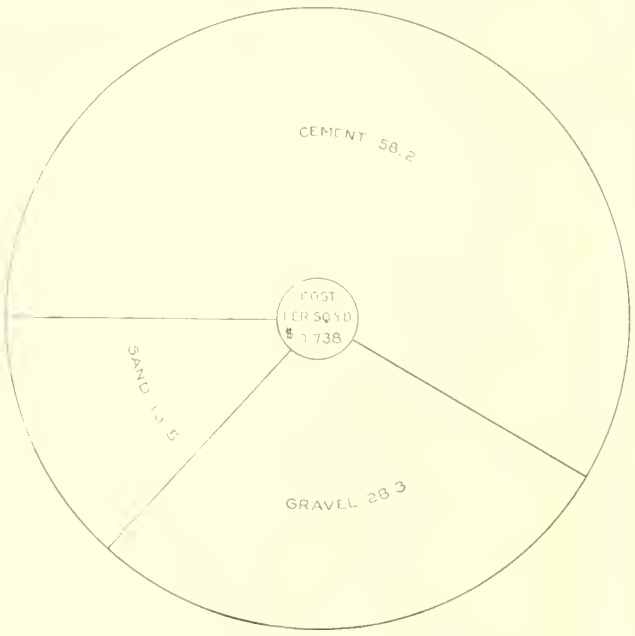


FIGURE 18.—MATERIALS COST DATA FOR WISCONSIN STATE-AID PROJECT 2916. COSTS ARE EXPRESSED AS PERCENTAGES OF TOTAL COST OF MATERIALS

same truck haul with the over-all efficiency of one contractor 83.1 per cent and that of the other 88 per cent. One job used local aggregate and the other commercial aggregate. Five sacks of cement per cubic yard were used on each job.

Further reductions in hauling, labor, and depreciation will probably be made this year in Wisconsin by the use

COST CHART

ORGANIZATION CHART

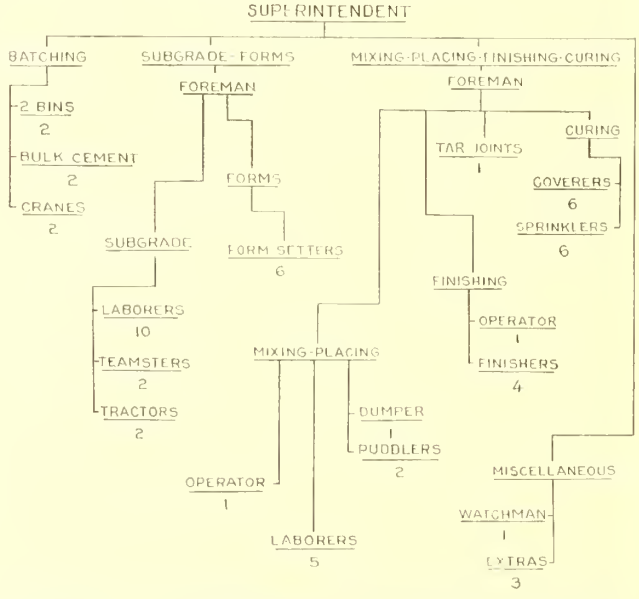


FIGURE 19.—LABOR COST DATA AND ORGANIZATION CHART FOR WISCONSIN STATE-AID PROJECT 2916. COSTS ARE EXPRESSED AS PERCENTAGES OF TOTAL COST OF LABOR

of a 33-cubic-foot batch and a 50-second mixing time after all solid materials are in the drum.

On these two Wisconsin jobs and on the job in Arkansas, all of which used separate sizes of coarse aggregates, 242 batches were sampled as they were dumped on the subgrade and tests made to determine the distribution of coarse aggregate, sand, cement, and water in samples taken from the four corners and the center of the batch. From these same batches beams and cylinders were also made. A total of 1,276 cylinders and 726 beams were made in these tests. All of these tests indicate that a high degree of uniformity was obtained not only within the batches but also between batches.

TABLE 25.—*Summary of time losses and their effect on production, for Wisconsin State-aid project 2926*

MAJOR DELAYS OCCURRING DURING AVAILABLE WORKING HOURS

Character of delays	Hours	Per cent
Rain	31.75	5.19
Wet grade	32.50	5.30
Moving during job	35.25	5.75
Material supply	26.25	4.28
Subgrade	8.00	1.30
Mixer trouble, mechanical	3.07	.50
Water supply	2.83	.46
Batcher	5.28	.86
Finishing machine	1.95	.32
Crane	5.50	.90
Joints	1.50	.26
Miscellaneous	5.85	.96
Total	159.73	26.08
Time paver actually operated	453.27	73.92
Available working time	613.00	100.00

MINOR DELAYS OCCURRING DURING TIME OF ACTUAL OPERATION

Character of delays	Hours	Per cent
Hauling supply	7.65	1.69
Hauling operation	4.31	.95
Dumping	6.25	1.38
Mixer trouble, mechanical	4.90	1.08
Mixer operation	8.46	1.86
Water supply	11.30	2.49
Joints and steel	4.93	1.09
Subgrade	3.60	.79
Miscellaneous	12.79	2.82
Total	64.19	14.15
Time paver operated at 100 per cent efficiency	389.08	85.85
	453.27	100.00

Over-all efficiency of paver, 88.0 per cent; total production, 116,347 square yards.

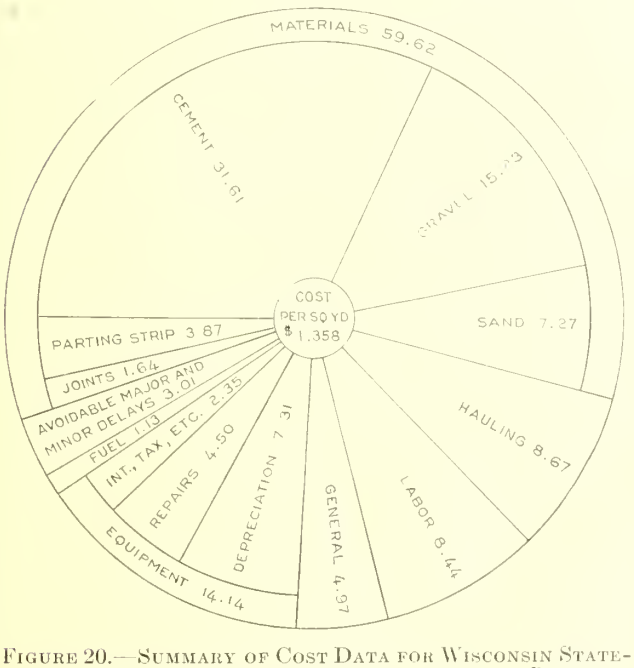


FIGURE 20.—SUMMARY OF COST DATA FOR WISCONSIN STATE-AID PROJECT 2916. COSTS ARE EXPRESSED AS PERCENTAGES OF TOTAL COST OF PROJECT

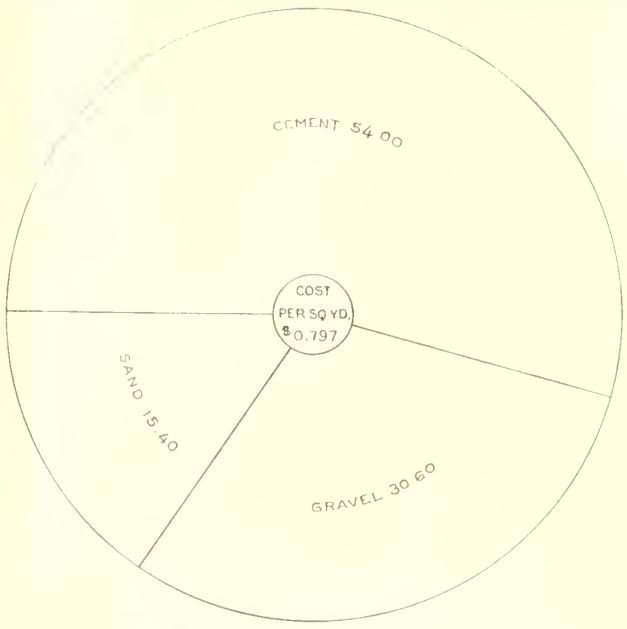


FIGURE 21.—MATERIALS COST DATA FOR WISCONSIN STATE-AID PROJECT 2926. COSTS ARE EXPRESSED AS PERCENTAGES OF TOTAL COST OF MATERIALS

CONCLUSIONS SUMMARIZED

From the data presented in the quality and production studies of 27E pavers in good condition the following conclusions seem reasonably clear:

1. The uniformity of the mixing and the resulting strength of the concrete are equally good for 35-cubic-foot batches as for 27-cubic-foot batches when mixed for 60 seconds.

2. The uniformity of the mixing and the resulting strength of the concrete are equally good for a 30 or 33 cubic-foot batch when mixed for 50 seconds as when mixed for 80 seconds.

3. Under present actual job conditions and customary auxiliary equipment, the most economical batch for the 27E paver seems to be 33 cubic feet with a mixing time of 50 seconds for all solid materials.

4. The large-sized batches, up to 35 cubic feet, and on grades less than 6 per cent, did not cause spillage, leakage, or clogging of either the old or the new pavers, and may be advisable when a mixing time longer than 50 seconds is required.

5. The mixers did not show any signs of breakage related to overloading from using the larger-sized batches.

6. Practically no increse in the present standard auxiliary equipment or labor set-up, nor any change in this equipment or labor set-up, is necessary to use a 33-cubic-foot batch mixed 50 seconds.

7. If oversanded mixes are still retained after the introduction of the use of aggregates of multiple sizes, the resulting concrete will not only cost more than it otherwise would but quality will be sacrificed. By using separated sizes intelligently a lower mortar ratio

(Continued on page 292)

RELATION BETWEEN THE STRENGTH OF CEMENT AND THE STRENGTH OF CONCRETE

By F. H. JACKSON, Senior Engineer of Tests, U. S. Bureau of Public Roads

ABOUT four years ago the Bureau of Public Roads began a series of laboratory tests for the purpose of determining the extent to which the strength of concrete may be expected to vary as the result of variations in the quality of the Portland cement used in its manufacture. At that time engineers were just becoming interested in the use of relatively rapid hardening or high early strength cements in concrete road construction. It was felt that the use of such cements would be justified for economic reasons through the earlier opening of highways to traffic, provided assurance could be had that the ultimate strength and durability of the concrete would not suffer. The standard requirements of the American Society for Testing Materials had not then been raised to the present values and the general level of briquet strengths shown by Portland cements was not as high as it now is.

Because of this recent trend toward the manufacture of cements of higher early strength, the results of these tests may not have quite the significance which they otherwise would have. However, it is believed that the data will be of some interest, first in showing to what extent briquet strengths reflect the corresponding concrete strengths developed by various Portland cements at different ages, and second, the extent to which variations in the quality of Portland cement affect the conventional water-cement ratio strength relation.

DESCRIPTION OF TESTS

Samples of eight brands of Portland cement were obtained from warehouse stocks in the Washington, D. C., market, and a series of concrete tests were made in which the attempt was made to eliminate every variable except the quality of the cement. A nominal 1:2:4 volumetric mix (dry rodded) was employed, using Potomac River sand and gravel and a sufficient quantity of water to give a slump of about two inches. The actual quantity of water was varied slightly, because of differences in the normal consistencies of the cements. However, the maximum difference in water-cement ratio due to this cause was so slight (amounting to only about 0.04) as to be of no significance as regards its effect on strength, and the water content may be said to have been substantially constant.

All specimens were fabricated and stored in accordance with American Society for Testing Materials standard practice. The briquets were stored in water in pans which were in turn stored in the concrete moist closet. The sand showed a fineness modulus of approximately 2.70 and passed all of the usual specification requirements. The gravel was graded uniformly from one and one-half inch down to one-fourth inch and was measured in three separate sizes.

The concrete was tested in compression, bending, and direct tension at periods of 7 and 28 days, six months, one year, and three years. Compression specimens were the usual 6 by 12-inch cylinders, transverse specimens 6 by 6 by 30-inch beams, tested as cantilevers and tension specimens 6 by 21-inch cylinders, tested by means of the gripping device developed in the laboratory of the Portland Cement Association.[1]

[1] Descriptions of the cantilever testing machine and the tension testing device appear in PUBLIC ROADS, vol. 10, No. 4, June, 1929, p. 74.

A complete series of the standard cement tests, including 20 sets of mortar briquets (three briquets to each set) was also made. This made it possible to test 12 briquets at each of the five ages indicated. Average results of all of the cement tests, with the exception of strength, are given in Table 1. The mortar tension tests for each cement for various periods up to three years and the corresponding concrete test results are given in Table 2.

TABLE 1.—*Routine tests of cements*

Cement	Time of set				Soundness	Fineness retained on No. 200 sieve	Normal consistency
	Initial		Final				
	Hrs.	*Min.*	*Hrs.*	*Min.*		*Per cent*	*Per cent*
A	3	45		[a]	Satisfactory	15.1	23.0
B	3	35	8	30	do	16.6	23.0
C	4	20	8	30	do	14.8	21.8
D	4	35	8	35	do	13.9	23.8
E	4	06		[a]	do	15.9	24.0
F	3	35	7	30	do	14.3	23.0
G	3	40	7	40	do	14.1	23.0
H	4	20	6	35	do	15.9	23.5

[a] More than 7 but less than 9 hours.

DISCUSSION OF RESULTS

The variations in strength obtained for a constant water-cement ratio, as shown by the results of 28-day compression tests given in Table 2, cover a range in strength from 2,005 to 2,775 pounds per square inch, as compared to an average for the eight brands of 2,414 pounds per square inch. This is a total range of 770 pounds, or 32 per cent of the average value. According to the general relation $S = \frac{14{,}000}{7^x}$, in which S represents crushing strength and x the water-cement ratio, the crushing strength corresponding to the water-cement ratio used (0.95) should be about 2,200 pounds per square inch. Based on this value, the concrete of highest strength showed a deviation of 26 per cent from the theoretical value, with the other concretes showing proportionately smaller deviations.

The average for the eight brands (2,414 pounds per square inch) was, however, close to the theoretical strength called for by the conventional formula.

These tests indicate that the maximum deviation in crushing strength at 28 days due to variation in the quality of the Portland cement, was approximately equal to the variation in strength which would be obtained by changing the water-cement ratio one gallon per sack of cement. The results corroborate, in general, such other data as are available, which indicate that the quality of the Portland cement may have a rather marked effect upon the 28-day strength of the concrete.

It will be of interest to compare the variations in concrete strength at 28 days with the variations in strength which were observed at later periods. An inspection of Table 2 shows about the same total variation in strength at each of the five ages at which tests were made, with a somewhat greater spread at the shortest and longest testing periods (7 days and 3 years) than for the intermediate periods. The relative order of the strength values did not, however, remain the same, certain of the cements which were high at the early periods showing comparatively low values

TABLE 2.—*Effect of quality of cement on the quality of concrete* [1]

Age	Cement	Briquet strength,[2] pounds per square inch	Strength of concrete in pounds per square inch: Flexure	Compression	Tension
7 days	A	325	261	1,349	144
	B	290	249	1,402	130
	C	265	211	1,092	126
	D	295	276	1,530	146
	E	310	265	1,830	160
	F	290	247	1,430	132
	G	265	230	1,299	112
	H	360	372	1,940	202
	Av.	300	264	1,484	144
28 days	A	375	391	2,160	227
	B	365	338	2,005	224
	C	345	330	2,100	198
	D	385	395	2,700	244
	E	385	401	2,775	222
	F	395	392	2,680	231
	G	350	384	2,428	212
	H	410	432	2,462	246
	Av.	376	383	2,414	226
6 months	A	405	540	3,149	229
	B	395	509	3,379	242
	C	395	563	3,861	240
	D	380	532	3,701	228
	E	340	533	3,560	249
	F	335	509	3,745	217
	G	400	541	3,706	222
	H	365	492	3,410	227
	Av.	377	527	3,561	232
1 year	A	405	482	3,539	244
	B	380	517	3,780	224
	C	400	519	4,086	233
	D	375	559	4,026	260
	E	355	523	4,064	235
	F	360	564	4,328	241
	G	410	564	4,225	238
	H	390	563	3,811	229
	Av.	384	536	3,982	238
3 years	A	375	549	3,490	251
	B	370	561	[3] 3,410	248
	C	375	583	4,140	286
	D	395	564	3,670	264
	E	355	540	4,170	267
	F	370	575	[3] 4,490	----
	G	345	554	3,650	235
	H	350	566	[3] 3,590	254
	Av.	367	562	3,830	258

[1] Mix 1:2:4 by volume; water-cement ratio, 0.95; consistency, 2-inch slump. Each value for modulus of rupture, average of 4 tests on 2 beams. Each value for compression, average of 4 tests, except as noted. Each value for tension, average of 4 tests on 2 cylinders.

[2] Tensile strength of 1:3 Ottawa sand mortar briquets. Each result is the average of 12 tests. Briquets stored in moist closet 24 hours; balance of time in water contained in pans stored in moist room.

[3] Average of 3 tests.

TABLE 3.—*Relative order of eight cements in mortar and concrete at various ages*

Age	Order	Tension tests of mortar briquets	Concrete tests: Tension	Flexure	Compression
7 days	1	H	H	H	H
	2	A	E	D	E
	3	E	D	E	D
	4	D	A	B	F
	5	F	F	F	B
	6	B	B	A	A
	7	G	C	G	G
	8	C	G	C	C
28 days	1	H	H	H	E
	2	F	D	E	D
	3	E	F	D	F
	4	D	A	F	H
	5	A	B	A	G
	6	B	E	G	A
	7	G	G	B	C
	8	C	C	C	B
6 months	1	A	E	C	C
	2	G	B	G	F
	3	B	C	A	G
	4	C	A	E	D
	5	D	D	D	E
	6	H	H	F	H
	7	E	G	B	B
	8	F	F	H	A
1 year	1	G	D	F	F
	2	A	A	G	G
	3	C	F	H	C
	4	H	G	D	E
	5	B	E	E	D
	6	D	C	C	H
	7	F	H	B	B
	8	E	B	A	A
3 years	1	D	----	C	F
	2	A	----	F	E
	3	C	----	H	C
	4	B	----	D	D
	5	F	----	B	G
	6	E	----	G	H
	7	H	----	A	A
	8	G	----	E	B

later. This is shown in Table 3, which gives the relative order of the eight cements for each testing period and for each type of test.

In general, it will be seen that, for the two earlier periods, 7 and 28 days, the briquet strengths of the various cements indicate quite definitely the relative order of the concrete strengths which will be developed by the same cements at the same ages. The only outstanding exception to this rule is cement A which shows a 7-day briquet strength quite out of line with the concrete strengths developed with this cement. At 28 days it will be noted that this cement has dropped to fifth place in briquet strength, which is about the order of strength in concrete for this cement at both 7 and 28 days.

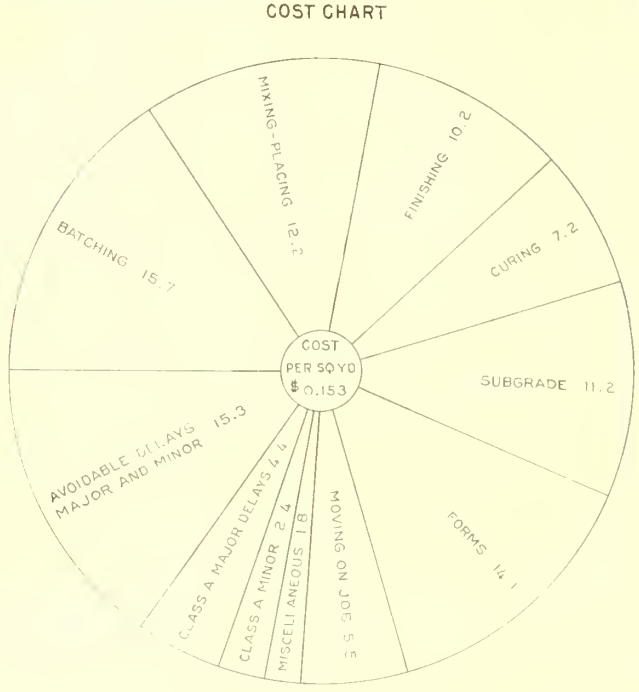
After the 28-day period practically all trends as regards the relationship between briquet strength and concrete strength disappear. This is true, not only of the 7 and 28 day briquet strength as compared to concrete strength at various periods, but is true also of the long time briquet strengths as compared to concrete strengths at the corresponding ages. Here again the data are in accord with other available information on the subject, which in general, indicates that at ages of six months and over, cements which give relatively low briquet strengths at 7 and 28 days, may show as high or higher concrete tests as those cements which give high 7 and 28 day test results.

From an examination of Table 2 it will be observed that there was a very definite retrogression in the strength of mortar briquets at three years, in all cases except that of cement D, and in some cases at earlier ages. This tendency is not reflected in any systematic way in the concrete tension tests, and we may, therefore, conclude that it was not caused by the particular cements used, but by some other factor, such as type of specimen, method of storage, etc., common to all specimens. In flexure also there was very little tendency toward retrogression in strength, except in the case of cements C, E, and A at one year and cement G at three years. These decreases may be accidental, although the same tendency appears in the tension tests in the case of cements C and E. On the other hand, none of these cements show retrogression in crushing strength at one year, although cements G, B, D, A, and E show a decrease at three years. In only one case (cement G) is there a corresponding reduction in the flexure and concrete tension strengths. It seems reasonable to assume that any marked tendency for a particular cement to cause retrogression in strength would be reflected in all of the concrete tests. It is believed that, in general, the reduction in crushing strength at three years was caused, not by the use of particular cements, but by some factor which can not be determined by the data available.

(Continued on page 292)

(Continued from page 289)

can generally be used and as a consequence the cement factor may be reduced.



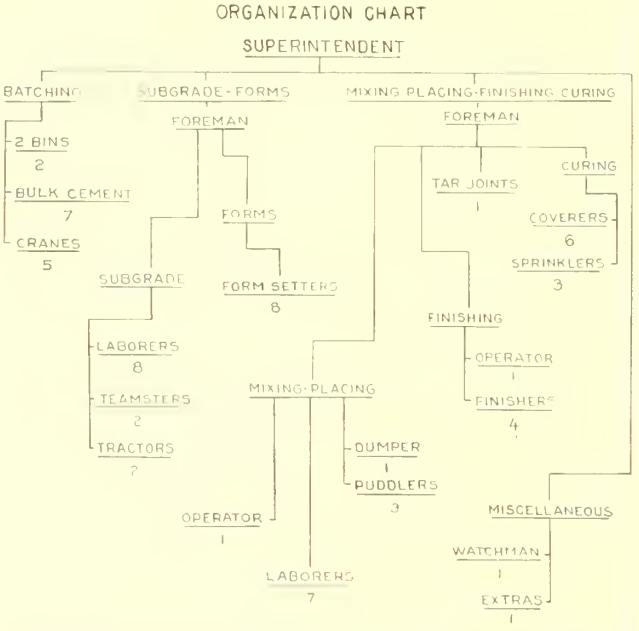


FIGURE 22.—LABOR COST DATA AND ORGANIZATION CHART FOR WISCONSIN STATE-AID PROJECT 2926. COSTS ARE EXPRESSED AS PERCENTAGES OF TOTAL COST OF LABOR

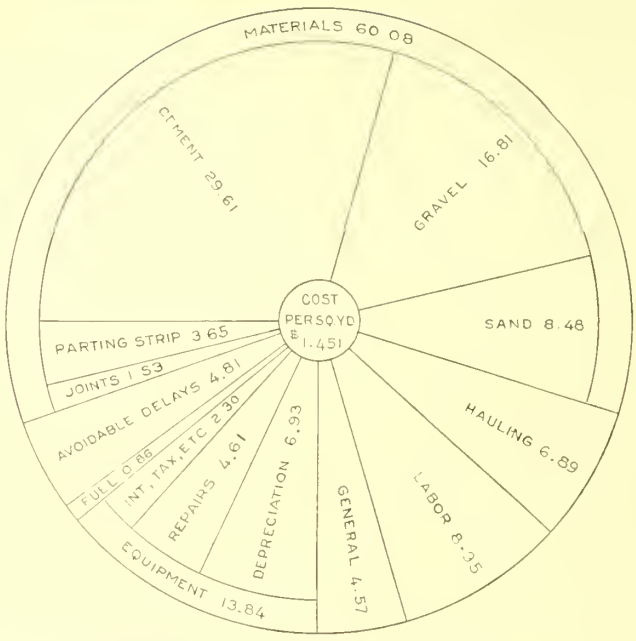


FIGURE 23.—SUMMARY OF COST DATA FOR WISCONSIN STATE-AID PROJECT 2926. COSTS ARE EXPRESSED AS PERCENTAGES OF TOTAL COST OF PROJECT

(Continued from page 291)

These tests substantiate the general conclusions which have been reached by concrete engineers to the effect that the 7 and 28 day briquet strengths of Portland cement are no measure of the comparative strengths of the concrete at later periods (say six months and over). The tests do indicate, however, that routine briquet strengths at 7 and 28 days measure, in a general way, the comparative strengths which will be developed in concrete at corresponding periods.

INDEX TO VOLUME 11 OF PUBLIC ROADS AVAILABLE

An index to volume 11 of PUBLIC ROADS, which includes the issues from March, 1930, to February, 1931, is now available for distribution, and copies may be obtained without charge from the Bureau of Public Roads, United States Department of Agriculture, Washington, D. C. Indexes to volumes 6, 7, 8, 9, and 10 have previously been published, and a supply of these indexes is still on hand. The index to volume 12 is now being prepared.

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Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets not to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.

Report of the Chief of the Bureau of Public Roads, 1925.

Report of the Chief of the Bureau of Public Roads, 1927.

Report of the Chief of the Bureau of Public Roads, 1928.

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265T. Electrical Equipment on Movable Bridges.

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Report of a Survey of Transportation on the State Highways of New Hampshire. (1927)

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Report of a Survey of Transportation on the State Highways of Pennsylvania. (1928)

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.

Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.

Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.

Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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*Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

DECEMBER 31, 1931

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal-aid allotted	MILEAGE Initial	MILEAGE Stage	MILEAGE Total	Estimated total cost	Federal-aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total		
Alabama	2,388.2	$ 1,777,588.89	$ 864,676.57	79.8		79.8						$ 5,379,923.57	Alabama
Arizona	1,117.0	3,562,907.58	2,343,687.11	145.2	64.7	209.9	$ 300,851.26	$ 226,518.42	22.8	19.8	42.9	1,600,099.18	Arizona
Arkansas	1,910.6	4,068,522.65	1,909,018.01	96.4	72.7	189.1	362,032.53	181,016.26	17.3		17.3	1,991,140.13	Arkansas
California	2,158.0	11,434,880.17	4,616,110.99	264.8	52.4	307.3	1,980,622.58	970,432.19	44.9	.5	45.4	2,721,765.29	California
Colorado	1,507.0	5,416,815.46	2,742,188.39	231.7	86.0	297.7	703,739.90	313,512.72	9.2	4.5	13.7	2,005,136.74	Colorado
Connecticut	282.7	3,627,221.97	1,279,199.52	28.7		28.7						706,266.13	Connecticut
Delaware	361.3	206,846.00	103,473.00	4.8		4.8	395,470.75	197,735.37	19.1		19.1	372,952.56	Delaware
Florida	557.4	5,946,262.29	2,785,791.65	171.4		171.4						3,046,471.31	Florida
Georgia	3,002.5	6,598,761.25	3,071,219.80	167.1	143.9	311.0	1,853,857.89	901,318.59	61.7	37.1	98.8	1,606,162.06	Georgia
Idaho	1,474.3	1,794,657.51	1,031,277.61	89.0	87.9	176.9	204,243.49	122,522.12	9.0		9.0	1,471,202.64	Idaho
Illinois	2,631.3	21,041,452.50	9,705,184.85	612.7	28.7	641.4	4,079,085.62	1,803,097.17	134.6	4.7	138.3	3,553,117.70	Illinois
Indiana	1,764.6	7,820,417.43	3,872,008.57	212.4		212.4	1,916,780.29	817,959.90	67.7	12.6	80.2	2,442,964.90	Indiana
Iowa	3,379.0	331,333.50	138,397.76		11.9	11.9	875,564.99	412,779.20	17.8	15.3	33.1	2,787,026.00	Iowa
Kansas	3,638.1	3,356,531.65	1,567,281.94	172.9	15.2	188.1	805,943.50	399,036.29	71.9		71.9	2,760,080.01	Kansas
Kentucky	1,878.8	2,144,969.67	965,595.15	130.5	4.0	134.5	80,102.42	40,051.21	5.5		5.5	2,031,876.08	Kentucky
Louisiana	1,638.6	7,319,502.67	2,857,785.52	117.7	10.6	128.3	1,295,507.93	68,270.07	.5		.5	1,644,337.95	Louisiana
Maine	706.0	1,804,346.15	789,080.76	31.4		31.4	37,646.34	14,085.00	.9		.9	1,304,944.37	Maine
Maryland	769.2	465,159.40	151,463.01	14.4		14.4	250,874.76	125,437.37	11.8		11.8	785,651.09	Maryland
Massachusetts	767.8	10,698,242.29	3,438,156.15	99.5		99.5	967,832.21	355,459.70	12.2		12.2	868,329.01	Massachusetts
Michigan	1,974.3	9,833,130.95	4,207,272.99	340.4	58.1	398.5	672,750.00	336,375.00	32.7		32.7	3,526,676.69	Michigan
Minnesota	4,291.3	2,047,501.22	779,785.45	20.7	58.8	79.5	6,679,825.80	2,146,085.76	119.4	172.2	291.6	527,121.61	Minnesota
Mississippi	1,809.7	3,462,678.29	1,715,852.10	166.7	60.8	227.5	446,856.30	200,118.74	2.2	17.3	19.5	5,487,995.36	Mississippi
Missouri	2,810.0	3,600,974.93	1,495,071.62	95.7	14.0	109.7	1,366,595.12	506,208.31	47.9	3.5	51.3	3,070,214.11	Missouri
Montana	2,602.8	5,641,169.72	3,157,553.44	440.5	22.8	463.3	461,551.57	261,257.48	74.0	1.4	75.4	3,199,372.54	Montana
Nebraska	4,110.8	6,282,652.45	2,987,340.32	235.7	90.1	326.8						2,620,855.46	Nebraska
Nevada	1,307.8	1,450,418.78	1,098,118.53	41.4	118.9	158.3						1,553,046.61	Nevada
New Hampshire	418.9	636,114.46	246,912.44	12.1	2.5	14.8						647,810.24	New Hampshire
New Jersey	574.1	6,233,030.43	2,156,623.50	67.4	.6	68.0						1,704,126.59	New Jersey
New Mexico	2,177.7	2,868,433.47	1,834,223.29	135.7	13.8	149.5	107,277.63	67,700.35	.3		.3	1,445,528.99	New Mexico
New York	3,204.0	14,783,200.00	6,219,226.00	317.3		317.3	2,479,500.00	963,025.00	49.6		49.6	4,784,623.59	New York
North Carolina	2,209.8	1,503,642.53	730,848.59	72.0		72.0	223,477.16	111,738.56	17.4	4.0	21.4	3,864,930.24	North Carolina
North Dakota	5,113.0	1,862,481.70	944,932.80	186.9	171.9	358.8	1,327,226.48	679,096.29	149.8	259.5	409.1	1,961,206.41	North Dakota
Ohio	2,914.7	7,403,122.99	2,579,944.47	114.1	29.8	143.9	1,108,207.63	547,514.39	9.6	.4	7.0	4,146,278.47	Ohio
Oklahoma	2,193.6	5,289,792.65	2,672,923.28	137.3	84.0	221.3	54,280.31	30,017.00	.1		.1	2,534,175.20	Oklahoma
Oregon	1,537.8	3,289,149.12	1,824,448.62	92.1	35.7	127.8	108,061.90	65,447.84	3.7	11.1	14.8	2,144,258.56	Oregon
Pennsylvania	2,991.4	4,776,384.09	2,173,477.80	66.5		66.5						5,155,863.62	Pennsylvania
Rhode Island	257.5	811,553.61	427,281.05	16.8		16.6	81,305.62	40,652.81	2.1		2.1	325,046.83	Rhode Island
South Carolina	1,987.2	2,898,736.33	1,258,506.90	63.0	53.5	116.5						1,369,843.00	South Carolina
South Dakota	4,035.9	2,851,923.36	1,578,779.43	174.8	176.3	350.1	317,827.24	180,943.81	35.1		35.1	1,416,989.47	South Dakota
Tennessee	1,663.7	848,884.19	422,357.41	24.2	8.7	32.9	437,058.09	202,974.45	13.6	6.4	20.0	3,870,857.58	Tennessee
Texas	7,581.9	13,504,192.79	6,069,551.28	672.0	140.9	812.6	1,827,496.02	819,884.59	138.7	53.4	190.1	7,325,508.50	Texas
Utah	1,218.6	650,176.86	452,772.23	69.6	.4	69.8	215,919.55	159,755.43	13.5	3.2	18.7	1,403,504.03	Utah
Vermont	339.1	104,832.58	52,345.06	4.3		4.3						547,165.83	Vermont
Virginia	1,821.9	3,371,853.57	1,588,186.92	150.9	29.3	189.9	168,304.09	83,152.04	5.4		6.4	2,086,417.78	Virginia
Washington	1,177.7	2,253,263.80	1,079,786.83	97.9	8.5	74.1	588,082.89	223,000.00	26.6		26.6	1,790,375.81	Washington
West Virginia	861.6	2,256,068.25	889,293.85	67.1	10.5	77.6	234,360.46	109,369.58	.7		.7	1,196,368.17	West Virginia
Wisconsin	2,647.6	3,684,198.61	1,402,097.78	139.3	34.4	173.7						2,716,712.05	Wisconsin
Wyoming	2,060.4	2,308,642.10	1,369,859.87	157.6	79.9	234.1	307,527.38	196,458.66		35.8	36.8	858,687.26	Wyoming
Hawaii	63.1	904,811.19	414,072.39	27.9		27.9	151,626.85	54,354.00	3.6		3.8	2,018,071.60	Hawaii
TOTALS	99,777.4	216,849,699.83	98,079,850.49	9,818.2	1,853.9	9,671.8	35,473,244.54	15,134,461.75	1,247.6	682.6	1,910.2	113,879,088.01	TOTALS

[1]The term stage construction refers to additional work done on projects previously improved with Federal-aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

U. S. GOVERNMENT PRINTING OFFICE: 1932

PUBLIC ROADS

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 12, NO. 12 ▽ FEBRUARY, 1932

Photo by Bert Huntoon, Bellingham, Wash.

MT. SHUKSAN AS SEEN FROM SKYLINE EXTENSION, MT. BAKER NATIONAL FOREST HIGHWAY

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 12, No. 12 February, 1932

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

SOME STUDIES OF DRILLING AND BLASTING IN HIGHWAY GRADING

By ANDREW P. ANDERSON, Highway Engineer, Division of Management, U. S. Bureau of Public Roads

POOR blasting is the rule rather than the exception in much of our highway grading work. This conclusion is based on a study of 71 power-shovel grading jobs on which solid materials requiring drilling and blasting were encountered in considerable quantities. On 43 of these jobs the blasting was decidedly deficient. On 13 of the jobs the material was fairly well broken, while on only 15, or slightly more than one-fifth, was the material so well broken as to permit rapid and fairly continuous shovel operation.

Since poor blasting is so common and materials which must be blasted before they can readily be handled by the power shovel are so frequently encountered in present-day highway grading work, the accumulated field records of the production studies made by the Bureau of Public Roads during the past six years have been reviewed in order to obtain as much definite data as possible in regard to the following points:

1. How and to what extent does the blasting affect the rate of shovel production?

2. What are the chief obstacles to adequate blasting—that is, such blasting as will permit a high rate of shovel production?

3. How and to what extent can these difficulties be overcome?

The rate of power-shovel operation depends on three factors, the average length of the dipper cycle, the amount of material handled or moved per dipper load, and the regularity or continuity of operation. Poor blasting usually means large rocks and also, very frequently, tight or even some unbroken ground. Large rocks can only be handled with much difficulty and at a very slow rate. Still further delays are often imposed on the shovel while the larger rocks are being "bulldozed" or while the unbroken ground is being reblasted. Figures 1, 2, and 3 illustrate the difficulties imposed on the shovel by poor blasting.

On the other hand, where the blasting has been so thorough that the largest dimension of the larger fragments or particles does not exceed but about one-half of the smallest inside dimension of the dipper, the rate of shovel operation can be practically the same as for operation in good common earth excavation. Figure 4 shows a cut in which adequate blasting made the work of the shovel easy. The average amount of pay material moved per dipper load, however, will nearly always be less when working in rock than when working in good common earth excavation because of the fact that rock generally takes on more swell from blasting and from being picked up by the dipper than does ordinary earth picked up by the dipper from its natural place in the cut.

Even with the most perfect blasting the rate of production in pay yardage can rarely, if ever, be expected to be as high as can be achieved in good common earth under similar conditions and with equal effort. The same rate of shovel operation can be attained, and, under favorable conditions, the same apparent or loose-volume yardage; but the actual or pay yardage measured in place in the cut will probably always be less for the rock than for good earth. Very sticky materials or earth mixed with many roots or bowlders may give much lower production than well-blasted rock, as to both size of dipper load and actual yardage per hour of operation.

RATE OF PRODUCTION REDUCED MORE THAN 50 PER CENT BY POOR BLASTING

As the thoroughness of the blasting decreases, the rate of production decreases rather rapidly. For all the jobs studied during the past three years the average operating cycle of 1 to 1¼-yard shovels working in materials classed as well to fairly well blasted rock was 23.5 seconds, while for the same size and type of shovel operating in poorly blasted materials the average shovel cycle was 33.5 seconds—an increase of over 42 per cent. Even worse conditions are frequently found, as, for example, on some very poorly blasted jobs not included in the above averages on which the average shovel cycle was 45 seconds—an increase of more than 85 per cent over well-blasted materials.

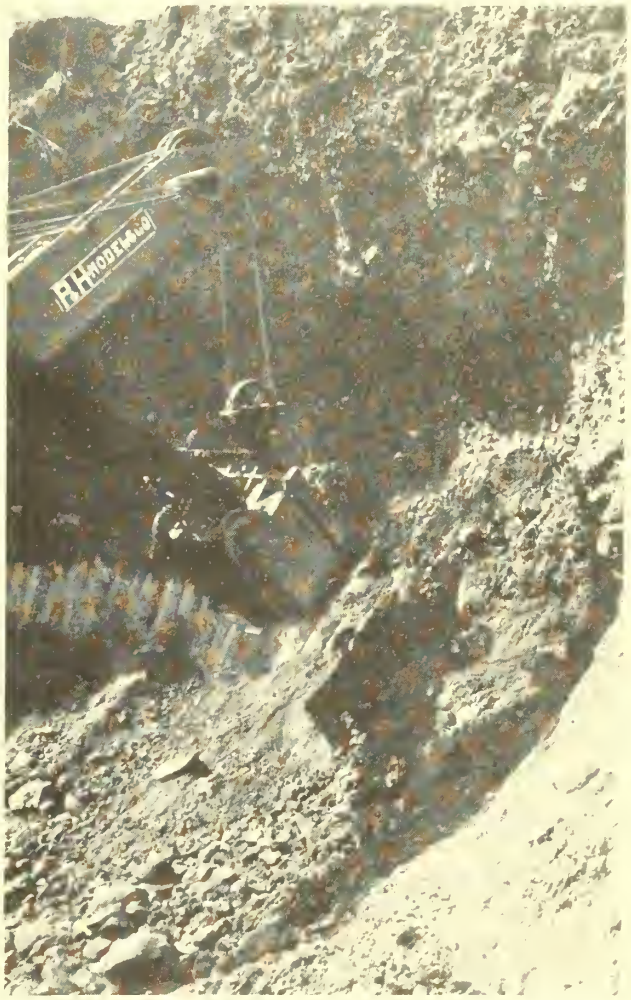


FIGURE 1.—TRYING TO MAKE THE SHOVEL TAKE THE PLACE OF DRILLING AND BLASTING IS A SURE WAY TO SHORTEN THE LIFE OF THE SHOVEL, POSTPONE THE COMPLETION OF THE JOB, AND REDUCE THE POSSIBLE PROFITS

The effect of poor blasting on the average quantity of pay material moved per dipper load is almost equally striking. For the 1 to 1¼ yard shovels the dipper load averaged 0.70 cubic yard of pay material in well-blasted rock but only 0.50 cubic yard in poorly blasted materials, a reduction of over 28 per cent. On a few jobs not included in the above averages, because of exceptionally poor blasting, the output was only about 0.4 of a cubic yard of pay material per dipper load for a 1⅜-cubic-yard dipper, a reduction of over 40 per cent. The average reduction in the rate of production for 1 to 1¼ yard shovels, not including exceptionally bad jobs, was therefore approximately 50 per cent, simply because of the decreased dipper load and the increased shovel cycle resulting from poor blasting of the material.

FIGURE 2.—A GOOD SHOVEL PLUS A GOOD OPERATOR CAN HANDLE ROCKS LIKE THIS, BUT THE COST PUTS THIS KIND OF BLASTING INTO A LUXURY CLASS WHICH NO CONTRACTOR CAN ANY LONGER AFFORD

This 50 per cent decrease in the rate of shovel production resulting from an increased shovel cycle and a decreased net dipper load does not include all the effects of poor blasting on the rate of production. More time is always lost on poorly blasted jobs than on jobs where the blasting is well done. This is brought out more fully in Table 1, which is a summary of the production studies during the past four years on 20 jobs using ⅞ to 1¼ yard shovels. Here it will be noted that the delays or individual stops due to one item alone, large rocks, were increased almost 50 per cent on the poorly blasted jobs, while the hourly production for the time the crew was on the job was reduced to less than one-half of that attained on the jobs on which the blasting was thorough.

TABLE 1. *Average rates of production and operation on 20 rock jobs*

Condition of blasted rock for handling by shovel	Average size of shovel	Average dipper load	Shovel production per hour			Percentage of working time lost by shovel due to large rocks
			Cubic yards	Dippers		
				Number	Cycle	
	Cu. yds.	*Cu. yds.*			*Seconds*	
Good	1.02	0.68	65.5	96.3	24.4	8.7
Fair	1.08	.49	38.7	79.7	32.2	14.5
Poor	1.12	.50	31.8	63.3	36.7	16.3

COST OF SHOVEL OPERATION VIRTUALLY INDEPENDENT OF RATE OF PRODUCTION

The reduction of some 50 per cent in production simply because the material was poorly instead of thoroughly broken would not be so bad if the cost of shovel operation could be reduced in about the same proportion. This, however, is impossible. The cost of shovel operation exclusive of the hauling is practically constant so long as the crew is out on the job regardless of whether production is high or low, and with the present prevailing equipment and methods of operation even the hourly or daily cost of the hauling equipment tends to become fairly constant and only partly dependent on the rate of production. Every decrease in output, therefore, tends automatically to register as an increase in the unit production cost.

The following example illustrates the point in question. The records show two 1-yard shovels each of which had an average operating cost of about $21 per hour for the items of loading, hauling, and dump operation. At one time during a full week while one of these shovels was working in well-blasted rock at an average rate of 67 cubic yards per hour the other was working in poorly blasted rock at an average rate of only 36 cubic yards per hour. It cost the contractor, therefore, 31⅓ cents per cubic yard to load, haul, and place the well-blasted material as against a cost of 58⅓ cents per cubic yard to handle the poorly blasted material, a difference of 27 cents per cubic yard. It seems probable that if a part of this 27 cents had been expended in obtaining a somewhat closer spacing of the drill holes and the use of a slightly larger amount of explosive a considerable saving would have been effected.

In this case, which was fairly typical of average conditions, the contractor, as is usually the case, was unable to reduce either his hauling equipment or his dump crew while working in the poorly blasted material. Poor blasting almost invariably gives rise to poor hauling conditions, the main results of which are a slow hauling speed and difficult dump operation. These conditions call for proportionately more hauling equipment. Furthermore, poorly blasted material is seldom uniform. There are therefore periods when the rate of production is high enough to demand the normal supply of hauling equipment and other periods when production is so low that most of the hauling equipment is idle. The wear and tear on all the equipment is excessive when working in poorly blasted material. Time losses and delays due to mechanical troubles are also more frequent. The question, therefore, naturally arises as to how and to what extent can these losses be prevented or reduced without incurring corresponding costs.

METHODS STUDIED FOR IMPROVING QUALITY OF BLASTING

It is recognized, of course, that under the many widely varying conditions encountered in the field and with the present knowledge of the art, not all blasts will produce the much desired thorough shattering of the material, and that no rules or formulas can be given which will fit all conditions. On the other hand, it is believed that the available data are sufficiently extensive and sufficiently clear to indicate certain general guides to procedure which should be of material value, first, when work is begun on a new job or a new cut which differs materially from those in which work has already been done, and second, in interpreting and utilizing the experience gained from each successive blast. Occasionally conditions will be encountered under which it will probably be cheaper to accept a reduced shovel output, even as expensive as that is, rather than to assume the expenditure necessary to accomplish thorough breakage of all of the material; but these cases seem to be relatively few and will be discussed more fully later.

In general, the data indicate that on a large majority of grading jobs involving solid rock, hard shale, or similar materials more thorough blasting is possible,

and that such blasting will tend to decrease rather than to increase the total unit cost of moving the material.

For a proper understanding of this matter it is necessary to keep in mind the fact that the drilling and blasting involved in highway grading is very different from that encountered in any of the other common lines, such as quarrying, mining, or tunneling. The roadway and grade are laid out without much reference to such factors as the dip or strike of the strata, or to the bedding planes, seams, or stratification of the material which is to be removed. Moreover, the greater portion of the rock work is encountered in sidehill cuts so that the depth and frequently also the character of the material to be blasted varies considerably from one side of the road to the other as well as along the axis of the cut. In fact, it is not uncommon to find cuts in which no two drill holes in a row are of the same depth, and no two rows are alike. The character of the rock, as a result of weathering, dip of strata, etc., may also in extreme cases be different for practically every hole. To develop standard methods and practices which under such varying conditions will always produce the degree of breakage and shattering necessary for fast shovel operation is naturally difficult and this superabundance of variables in the conditions to which any rules or methods must be adaptable is probably the main reason for the present prevalence of poor blasting in highway grading work.

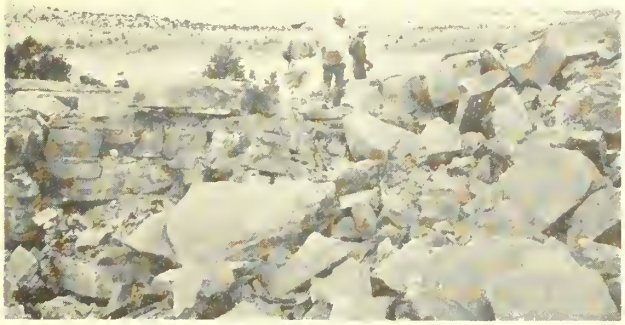


FIGURE 3.—UNDER AVERAGE HIGHWAY CONDITIONS POOR BLASTING SUCH AS THIS IS LIKELY TO INCREASE THE COST OF HANDLING MATERIAL AS MUCH AS 25 CENTS PER CUBIC YARD

LITTLE PROGRESS MADE IN IMPROVING METHODS

While notable progress has been made in effecting higher rates of production in practically all other lines of highway work, the rate at which drilling and blasting is handled has shown no appreciable progress during the past six years, for which the bureau has available definite records based on production studies on a large number of going projects. These data, while not sufficiently extensive to be entirely conclusive, are nevertheless sufficient to indicate that for identical equipment there has been during this period no material increase in the efficiency with which the drills are operated, in the footage of holes drilled per hour of actual operation, or in the rate of production of the shovel while handling blasted material. Nor is there any substantial evidence of a definite decrease in the time losses in drill operation, in the amount of either drilling or explosives required per cubic yard of pay excavation, or in the time losses imposed on the shovel by the operations of drilling and blasting. However, the trend toward the use of larger shovels has been rather marked and the proportionate loss of time has therefore been somewhat reduced.

From these indications one might argue that further progress in handling rock excavation in highway grading work is impossible or at least improbable; but, while the records of past performance are far from inspiring, they are not as hopeless as these generalities would indicate. Out of the 71 jobs there were 15 which attained practically as high a rate of production in rock as is usually secured in common excavation. One group of 1 to 1⅛ yard shovels working in material classified as "good to fair common" operated on an average cycle of 21 seconds while moving 93,000 cubic yards of material, while another group of shovels of the same size working in material classified as "rock, well blasted, very few pieces too large for dipper" also operated on an average cycle of 21 seconds while moving 48,000 cubic yards of material.

FIGURE 4.—ALMOST ANY ROCK, WHETHER STRATIFIED OR MASSIVE, AND NO MATTER HOW HARD OR TOUGH, CAN BE SO THOROUGHLY BLASTED THAT IT CAN BE HANDLED BY THE SHOVEL AT ABOUT THE SAME RATE AS GOOD COMMON. THIS IS THE SAME KIND OF ROCK AS IS SHOWN IN FIGURE 3

The average dipper load of the shovels working in common was 0.66 cubic yard of pay material while for those working in the well-blasted rock it was 0.61 cubic yard. Thus, for the time these shovels were actually digging, the rate of moving cubic yards of pay material in well-blasted rock was 92 per cent of the rate maintained by the shovels working in good common. But, probably because of a somewhat better supply of hauling equipment, the time losses on the rock jobs were about 6 per cent less than where good common was being handled, so that the actual production per hour on the rock jobs was only 2 per cent below that attained on the earth jobs. Based on the total time the crews were on the job, the actual production of pay yardage was 57.5 cubic yards per hour for the shovels working in good common and 56.4 cubic yards per hour for the shovels working in well-blasted rock. The ordinary time losses or delays, while rather unusually high, were very nearly the same for both sets of shovels, 49 per cent for those working in common excavation and 46 per cent for those working in rock. In both cases the major portion of these time losses was due to insufficient hauling equipment.

For these reasons it seems necessary to examine a little more closely the conditions which exist in drilling and blasting for highway grading work. In the first place, we find that on the great majority of jobs on which rock is encountered, it is usually covered with more or less overburden so that advance determination of such important factors as the exact amount, char-

acter, and condition of the rock, the dip of the strata, location of seams and bedding planes, etc., is difficult. Sometimes considerable rock is encountered where none had been expected. As a rule, the depth of rock to be removed in these cuts is comparatively small, seldom above 20 feet, on many jobs rarely over 4 or 5 feet, and frequently only a foot or two. The limitations placed on either over or under breakage on the side slopes, and frequently even in regard to the grade line, are rather numerous.

Careful consideration must also be given to preventing injury to persons, passing traffic, and neighboring improvements. The shooting must, therefore, as a rule, be as light as possible. But light shooting and thorough breakage of the material can only be achieved through correct depth and placement of drill holes which have been properly loaded with the right amount of suitable explosives and then properly fired. Even heavy shooting frequently fails to break up a hard or tough material properly unless the charges have been both placed and spaced correctly. (See fig 5.) Success

FIGURE 5.—IT TAKES MORE THAN GOOD INTENTIONS TO BREAK HARD AND TOUGH ROCK. THE BLAST JUST FIRED DID LITTLE MORE THAN BREAK THE MATERIAL INTO LARGE BLOCKS, EACH OF WHICH NOW FORMS A SEPARATE PROBLEM

can be attained only through a correct coordination of all of these factors while a substantial error in any one is almost certain to result in a costly failure. Since drilling is usually the most expensive item in the type of rock work encountered in highway grading, while improper location, spacing, or depth of the drill holes is also the most frequent cause of poorly broken material, we shall discuss this item in more detail.

METHOD SUGGESTED FOR SPACING DRILL-HOLES

Figure 6 shows a part of a rock cut where two rows of holes have been drilled. It will be observed that the holes are staggered. The relations between the spacing of holes, depth of cut, and depth of drill hole below grade are expressed as follows:

a = Ratio of depth of cut to spacing of holes across cut.

b = Ratio of depth of cut to spacing of holes from face, or of rows across cut.

c = Coefficient of depth of drill hole below grade. Its value, for most materials, lies between one-fourth and one-half, except in very shallow holes.

d = Depth of cut at drill hole.

ad = Distance between holes across cut.

bd = Spacing of rows or distance of holes from face of cut.

cad = Depth of drill hole below grade to which rock is to be removed.

$d + cad$ = Total depth of drill hole.

In this arrangement the drill-hole spacings parallel and perpendicular to the face of the cut are not equal, but each is a function of the depth. The depth drilled below grade is a function of the spacing rather than the depth.

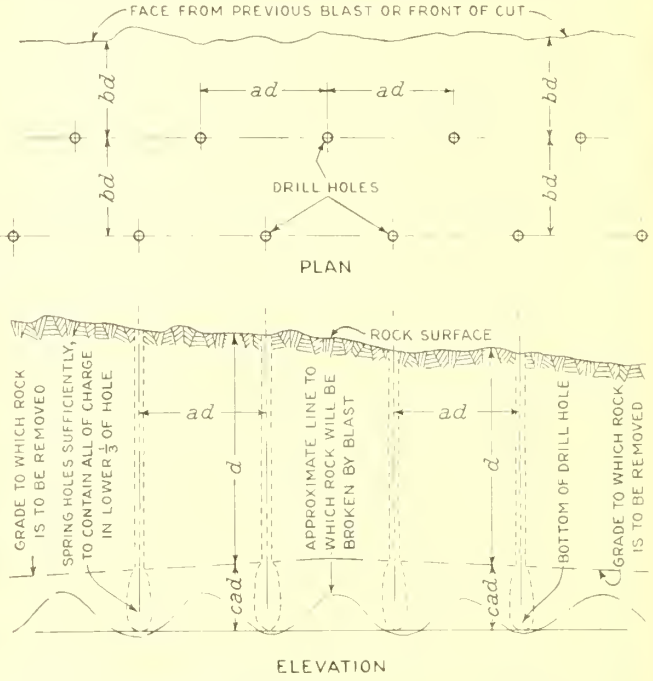


FIGURE 6.—SUGGESTIONS FOR PLACING DRILL HOLES. DIAGRAM SHOWS PART PLAN AND PART ELEVATION OF NEARLY LEVEL THROUGH CUT

A review of all the field records for jobs working in reasonably homogeneous material shows that the spacing between the holes in a row across the cut averaged about three-quarters of the total depth and usually varied from one-half to slightly more than the depth of the holes. The spacing of the holes from the face or between rows averaged slightly less than the spacing between the holes in a row. In such materials as ordinary shale, disintegrated granite, and hardpan the spacing in deep cuts sometimes equaled twice the depth of cut but was usually about one to one and one-half times the depth of cut. However, there was a rather strong tendency to adopt and maintain a fairly constant spacing for any given cut regardless of the variation in the depth of individual holes.

Too frequently the exact placement of a hole was determined by the existence of a good place to drill rather than by the relative distances to adjoining holes. The depth drilled below grade was most frequently an arbitrary constant for any given cut with but little or no variation for different depths of the individual holes. Thus, on one job on which the depth of cut varied from 2 to 10 feet and the spacing from 2½ to 6 feet, the depth below grade was always 2 feet. On a second job on which the depth of cut varied from 5 to 20 feet and the spacing from 4 to 10 feet, the depth below grade was always 2 feet. On a third job in disintegrated granite with cuts from 5 to 24 feet and the spacing about equal the depth, the depth below grade was always 3 feet, while on a fourth job with cuts mostly 10 to 20 feet deep and a spacing of about 12 feet between holes and 8 feet between rows, the depth below grade was always 4 feet. A further consideration of Figure 6 will help to illustrate why such practices are likely to result in improper fragmentation or shattering of the material, and high unit costs.

UNIT COST HIGH FOR SHALLOW CUTS

If the distances between holes and rows as well as the depth of the holes are given in feet with the designations as indicated in Figure 6, we note that the volume of material above grade line may be expressed in cubic yards for each hole as:

$$V = \frac{abd^3}{27};$$

while if D be used to designate the linear feet of drilling required per cubic yard of pay material, we have:

$$D = \frac{27\ (1+ac)}{abd^2}$$

Thus, within those limits in which the ratio of the spacing of holes and rows as well as depth below grade holds true, the volume of pay material per drill hole varies directly as the cube of the depth of the cut and the amount of drilling per cubic yard of pay material varies inversely as the square of the depth of the cut. The amount of explosives required per cubic yard of any given material, however, varies only slightly with the depth of the hole. These facts help to explain why the unit cost of removing shallow cuts of hard rock is so high.

If we consider, for example, four cuts of the same kind and character of uniform solid rock, the first of which is 2 feet deep, the second 4 feet, the third 8 feet, and the fourth 16 feet deep, in which both rows and holes can be spaced at three-quarters of the depth of the cut and the depth to which the holes must penetrate below the grade is one-fourth of the spacing between holes, then the amount of drilling, in linear feet, required per cubic yard of pay material for each cut is as follows:

2-foot cut	14.25
4-foot cut	3.56
8-foot cut	.89
16-foot cut	.22

Thus, if the cost of drilling be 30 cents per linear foot, as is frequently the case, then the cost of drilling alone is $1.07 per cubic yard of pay material for the 4-foot cut, $0.27 for the 8-foot cut, $0.07 for the 16-foot cut, and the entirely prohibitive figure of $4.27½ for the 2-foot cut. Even in the case of the 4-foot cut, the cost of the drilling alone is higher than the price frequently bid for moving solid rock, while on the other hand in the case of the 16-foot cut the unit drilling cost becomes but a relatively small item in the total cost of moving the material.

PROCEDURE MUST BE VARIED TO FIT ACTUAL CONDITIONS

The amount of explosives required per cubic yard of material tends to remain much more nearly constant, increasing only slightly as the depth of the holes decreases. Both theory and practical experience indicate that in shallow cuts it is cheaper to drill proportionately much deeper below grade than is advisable in the deeper cuts. This, of course, requires more explosives in moving nonpay material but decreases very materially the unit cost per cubic yard for the total combined operations of drilling and blasting. Experience has also shown that in very deep cuts of tough, very blocky, or nonuniform rock thorough breakage can not readily be accomplished if the holes are spaced as far apart as theory would indicate to be possible. While in most rockwork encountered in highway grading the spacing both between holes in transverse rows and of rows from the face or from each other bears some fairly definite relation to the depth of cut in that particular material, there is both an upper and a lower limit fixed largely by considerations of cost and the difficulty of effecting the proper degree of fragmentation. These limitations can be expressed as follows.

The spacing of drill holes both from the face and from each other should be proportioned to the depth of cut at that point, except that only in the most exceptional cases where is it impractical to drill the holes at least 4 feet deep need the spacing of rows ever be less than 3 feet from the face or between rows. For most rock the spacing between holes need never be less than 4 feet, provided that the total depth of the hole is not less than 4 feet and penetrates at least a foot and a half below the level to which the material must be removed. In very tough or blocky material, a spacing of over 10 to 12 feet will usually require the drilling of secondary holes to about one-third to one-half the depth of the cut in the middle of the areas formed by a hole at each corner. These secondary holes, as well as any supplementary charges in the stem of the main holes, are then fired simultaneously with the main charges. This method should give good breakage for cuts up to 20 feet, which probably is close to the limit of the present economic depth of a single lift in most rock materials. In sidehill work considerable care must be taken in arranging the layout of the drill holes in order to prevent a too wide spacing for the shallow holes.

As has already been indicated, it is usually found advisable to drill the very shallow cuts considerably below grade in order to permit a wider spacing and so reduce the drilling cost. While a strict adherence to the rather general practice of spacing the holes three-quarters of the depth of the cut would, in a 2-foot cut, require over 14 feet of hole per cubic yard of pay material, according to the formula given above, the drilling of 4-foot holes 4 feet apart and 3 feet from the face would reduce the drilling to about 4.5 feet of hole per cubic yard of pay material. The total amount of powder required per cubic yard of pay material would probably be about doubled, but this would be largely compensated for by the saving in blasting caps, wiring, and time and trouble of loading. Where hard shooting is permissible the total cost of drilling and blasting a cut as shallow as 2 feet can be still further reduced by additional increases of the depth and spacing of the holes and the amount of explosives. This is especially true for materials which are not very difficult to fragment or shatter properly. Increasing the depth of the holes in the above 2-foot cut to 5 feet and the spacing to 5 and 4 feet, respectively, would require only about 3.38 feet of drilling per cubic yard of pay material, a saving of about 1.1 feet of drilling at the cost of an increase of about one-fourth to one-half pound of explosives per cubic yard of pay material. Figure 7 illustrates the difficulties of drilling and blasting in shallow cuts of hard rock.

On a few jobs it was found advantageous to use two different depths of hole. For example, for one blast the first holes were drilled about 18 feet deep (3 feet below grade) with a spacing of 12 feet across the cut and about 11 feet between the rows which were not staggered. A second set of holes was then drilled in the center of the squares formed by the four holes to a depth of about 7 feet, or not quite through the hard top strata which rested on a thin seam of softer material.

The deep holes were sprung and the principal charge placed at the bottom with a smaller charge in the stem above the soft material. The shallow holes were only lightly sprung so that the charge occupied slightly more than one-half the depth. The explosive was 40 per cent dynamite and all charges were fired simultaneously.

FIGURE 7.—SHALLOW CUTS IN HARD ROCK ALWAYS PROVE EXPENSIVE. ALTHOUGH THE CONTRACTOR SPENT $2 PER CUBIC YARD ON DRILLING AND BLASTING IN THIS CUT (MOSTLY 2 TO 4 FEET DEEP), THE ROCK WAS STILL SO POORLY BROKEN THAT THE COST OF GETTING IT INTO THE FILL ADDED OVER ANOTHER DOLLAR

DRILLING COSTS AND RATE OF PRODUCTION DISCUSSED

On typical projects the average cost of drilling in fairly hard uniform rock with the customary jackhammers (see fig. 8) was nearly 30 cents per linear foot of finished hole. Largely because of the varying nature of the rock the drilling cost per foot varied over a wide range and occasionally reached about double this figure. One rather extensive job on which the average depth of the holes was 5 feet with most of the cuts from 2 to 5 feet deep, required 2.85 linear feet of drill hole per cubic yard of rock at a cost of $1.60 per cubic yard or 56 cents per linear foot of drill hole for the drilling alone. On the other hand, on some jobs involving mostly deep holes the drilling per cubic yard was sometimes as low as 0.1 foot. There is no definite indication that the net hourly rate of drilling is appreciably faster for shallow than for deeper holes, except in materials which give trouble from sticking or broken drills. The slightly larger time losses in the deeper holes from changing steel, blowing holes, etc., seem to be largely compensated for by the fewer moves until a depth of about 20 feet is reached, after which the rate slows up. All but five of the rock jobs studied used air-driven jackhammer drills weighing about 70 pounds each and all statements and conclusions are based on the use of drills of approximately this size and type.

The rate of drilling varies with the material, the type of equipment, and the management. On some individual jobs over 60 per cent of the time the drilling crew was out on the job was unproductive. The rate of production per elapsed hour was therefore low on these jobs, regardless of the kind of material. Table 2 shows the average rates obtained on all the jobs for which complete data were available, the net rate while the drill was actually in operation, and the causes of the more persistent delays.

The materials listed in Table 2 have been grouped in accordance with their apparent ease or difficulty of drilling as indicated by these studies rather than according to the usual classification. The table shows that while the average actual cutting rate of the drills in some materials was nearly 75 per cent more than in others, the actual production of linear feet of completed drill hole per hour during the time the crew was out on the job was much more nearly uniform and the maximum difference in actual production of finished drill hole per drill was less than 35 per cent.

TABLE 2.—*Average rates of production and time losses for drilling in various materials; 60 to 70 pound jackhammers used in most cases*

Item	Sandstone, hard shale, hardpan, disintegrated granite	Granite, porphyry, felsite, trap, basalt	Hard limestone, stratified but of uniform hardness	Limestone, stratified, with clay seams	Weighted general average of studies
	Feet	*Feet*	*Feet*	*Feet*	*Feet*
Feet drilled per hour, total study time	13.8	14.8	15.5	11.5	14.2
While drill was in operation	36.2	28.9	24.2	21.2	26.1
DELAYS DURING STOP-WATCH STUDIES[1]					
Change drills	12.1	9.8	7.1	7.8	8.9
Clean or blow holes	23.4	10.0	4.0	14.0	11.1
Steel stuck or broken	15.8	7.6	5.3	5.3	7.3
Move to new hole	2.5	4.9	7.2	3.4	4.0
Springing holes		1.5	1.0	1.7	1.4
Operator	3.2	4.4	4.7	4.0	4.0
Mechanical trouble	.7	2.2	3.2	2.0	2.0
No steel	.2	3.8	1.7	.5	2.6
Miscellaneous	4.1	4.4	1.9	7.1	4.3
Total lost time while out on job	62.0	48.6	36.1	45.8	45.6

[1] Delays are tabulated in percentage of available working time

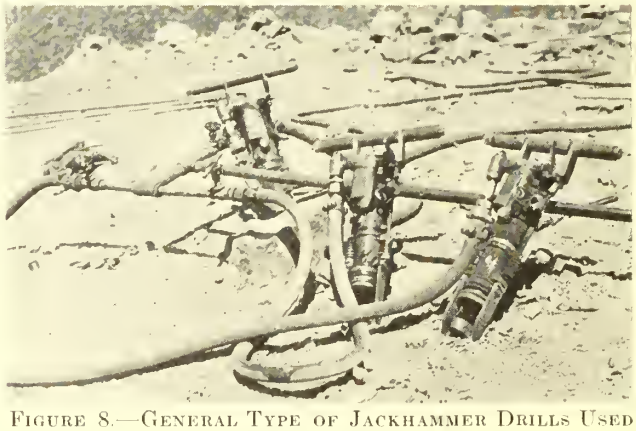


FIGURE 8.—GENERAL TYPE OF JACKHAMMER DRILLS USED IN HIGHWAY WORK

The time losses or delays in the easily drilled rocks of the first group were so large as more than to counterbalance the gainful effects of easy drilling. The chief delays were due to cleaning the holes and stuck or broken drills. These causes accounted for about two-thirds of the total time losses in the soft or decomposed materials, as against one-third for the same items in the igneous rocks of the second group. The poor showing in regard to actual cutting rate in the stratified limestones was largely due to the caution continually exercised by the drill operators to avoid crooked holes and stuck drills. That such caution can be made to pay is evident from the fact that while the cutting rate in hard stratified limestone was 25 per cent less than for the rocks of the first group, the actual production in finished drill holes was over 12 per cent greater. The generally rougher country in which the igneous rocks are usually encountered is reflected in the difficulty of keeping the drill operators supplied with steel. For the igneous group 3.8 per cent of the otherwise available

working time was lost on this account, as against a maximum of 1.7 per cent for any other group.

AIR PRESSURE AND QUALITY OF DRILL STEEL IMPORTANT

There are apparently several other conditions which influence the rate of drilling but upon which definite data are difficult to obtain. Thus, the rate at which the drill will cut in a given rock depends on the air pressure at the drill, the type and condition of the drill bit, and the hardness of the cutting edge or the length of time that the cutting edge remains fit for rapid cutting.

High air pressure is not of itself a guarantee of a high drilling rate for the average job. In hard rock a working pressure of over 100 pounds per square inch at the drill is very likely to cause excessive drill breakage. In a soft, seamy, or highly stratified rock with a large angle of dip, working on full open throttle at this pressure is likely to cause excessive sticking. In rock of this kind a working pressure above 100 pounds per square inch at the drill is not necessary. For very hard rock the present knowledge and facilities for obtaining the proper degree of hardness and toughness in the sharpened steel to withstand this pressure are not as yet available to the average highway job.

More attention should therefore be given to perfecting both the steel now used for drills and the methods of heat control and manipulation employed in the drill-sharpening outfits commonly found in highway work. Not only is there great need for a drill which will permit advantage to be taken of the higher cutting rate in hard rock which can be achieved by the use of higher air pressure, but there is also need of a more adequate drill steel or a means of hardening and tempering present steels so that they will better maintain their cutting edges and proper gage sizes.

Table 3 shows the effect of air pressure at drill on rate of cutting hard granite and hard stratified limestone. The stratifications, however, were all practically horizontal and therefore did not add to the difficulty of drilling.

TABLE 3.—*Effect of air pressure at drill on rate of drilling with 70-pound jackhammer in good condition. Holes 8 to 15 feet in both studies*

Working pressure at drill	Rate of drilling while drill was in actual operation	Number of 1-hour studies
	HARD LIMESTONE WITH HORIZONTAL STRATA	
Pounds per square inch	*Feet per hour*	
56	11.0	1
60 to 70	23.8	22
70 to 80	22.8	41
Above 80	28.9	2
	HARD GRANITE	
45	1.5	1
50	8.7	3
60 to 70	13.9	6
75 to 87	21.8	5

On a great majority of the jobs studied the working air pressure at the compressor ranged from 80 to 100 pounds per square inch. Thus, only one job was found which used a regular working pressure of 125 pounds and only one job using a regular working pressure below 70 pounds per square inch. But the pressure at the compressor may be quite different from that at the drills. In general, too little attention is given to the size or condition of the pipe and hose lines. Air leaks were numerous in most lines. Gage readings on one job showed an average loss of 1 pound of pressure for each 16 feet of ¾-inch hose or ¾-inch ordinary iron pipe line when supplying one drill at about 75 pounds pressure. However, but few jobs were found with a pipe line as small as this. The more common size was 1 inch for the pipe, although the use of ¾-inch hose was fairly common. A few jobs on which four to six drills were in regular use from a common pipe line used 2-inch pipe, while one job had a 2½-inch line. Pipe lines as long as 1,200 feet were found in use. Needless to say, pipes of this length must have very tight unions and be of sufficient size for the number of drills to be supplied, or the working pressure will be very low.

TIME LOSS IN UNPRODUCTIVE OPERATIONS DEPENDS ON OPERATING SKILL, EQUIPMENT, AND CHARACTER OF ROCK

On present jobs from 7 to 12 per cent of the working time is lost in changing drills, although it is known that the practice of using each steel until its full depth has been reached is far more common than that of changing the steel when it has become dulled to such extent that the rate of penetration is materially reduced. Thus, the common practice is to cut the steel into lengths which are multiples of 1½ or 2 feet, the latter being the most common. But hard and difficult rock will often dull the steel so much in drilling 1 foot that the time required for the following 6 inches is equal to that required for drilling the first foot. About the only times regular changes are made before the drill length has been attained is when the drills regularly lose so much gage that the following drill will not work readily. And even these conditions are usually blamed on the blacksmith, who is importuned to increase the die.

The studies show that under normal field conditions the drill is cutting rock only from 35 to 60 per cent of the total time the operator is on the job. The rest of the time is largely consumed in performing a number of necessary but unproductive operations, such as changing steel, blowing the hole, moving to the next hole, oiling the drill, etc. Frequently additional operations become necessary or are added to the duties of the drill operator, such as removing a stuck steel, clearing overburden, bringing up steel, or springing the drilled holes. (See Fig. 9.) The amount of time consumed in performing each of these operations varies with a number of factors, among which the skill and experience of the operator and the kind, character, and nature of the rock play a very important part. The time studies show that some operators regularly consume at least twice as much time in changing steel as do some others, while in certain kinds of rock the unskilled or inexperienced operator frequently loses several hundred per cent more time in extricating stuck steel than his more skilled and experienced coworker. The amount of time lost in performing the various necessary unproductive operations is also dependent on the character of the rock. In rock in which the drill will cut at the rate of a foot in two minutes, the number of steel changes per hour will ordinarily be about twice as many as in a rock in which the drill will only cut at the rate of a foot in four minutes. On the other hand, in soft rock more time is usually required for such operations as blowing or cleaning the hole and from steel getting stuck, while in very hard rock much time is frequently lost from broken steel, especially if the air pressure at the drills is too high.

The actual time in which each individual operation can be performed also varies with several factors. For example, aside from the skill of the operator, the time required to exchange steel varies with the depth of the hole. The following is a typical set of stop-watch readings of the exchange time for an excellent operator while drilling a hole 15 feet deep in fairly hard but uniform granite: 20, 22, 25, 32, 40, 40, 47, giving an average exchange time of 32 seconds for the hole. The first and second exchanges of steel are frequently made

FIGURE 9.—IN SOME ROCK AN INEXPERIENCED OR CARELESS OPERATOR WILL SPEND MUCH OF HIS TIME REMOVING STUCK STEEL. POOR SHARPENING EQUIPMENT OR A CARELESS BLACKSMITH WILL INTENSIFY THIS TROUBLE

in 15 to 20 seconds and single individual readings as low as 12 seconds are on the records. The increase with depth is rapid, and a 20 to 25 foot steel is seldom exchanged in less than a minute with the average time nearer 2 minutes. If the steel happens to stick or bind in the chuck, as is frequently the case, the exchange time for that particular change may readily be doubled or even trebled. This is the reason why the records of many thousand stop-watch studies show an average exchange time of about 45 seconds for holes from 12 to 15 feet in depth, while the individual readings range from 12 seconds to somewhat over 6 minutes, and the percentage of the operator's time consumed in exchanging steel varies from less than 3 per cent to over 25 per cent with an average of 8.9 per cent for 20 fairly large jobs. (See Tables 2 and 3.)

The contractor who has much rockwork should therefore find it profitable to give considerable attention to the training of the drill operators to perform all necessary operations in the shortest possible time. The drilling crew is really only productive when the drill is actually cutting rock. The various operations which interfere with actual production should therefore be reduced to the lowest possible limit. The contractor's interest must not end with simply insuring a high utilization of the available time. He must also see that the utilized time is converted into the greatest possible number of linear feet of properly placed and spaced holes. This requires the constant maintenance of adequate air pressure at the drills, together with an abundant supply of properly sharpened and correctly tempered steel of the right lengths within easy reach of the operator. One of the surest ways to make certain a low production from the drilling crew is to permit any one of the following to occur: Low air pressure at drills, too soft or too hard steel, poorly sharpened or poorly gaged steel, or requiring the drill operator to bring up his own steel or to move a heavy overburden in order to locate the holes. Under many conditions any one of these can readily reduce production more than 25 per cent, and if two or more are allowed to occur simultaneously production may readily drop to one-half of what would otherwise be possible.

AMOUNT OF EXPLOSIVES USED VARIES WIDELY

The amount of explosives required per cubic yard of identical pay materials is generally larger in highway grading than in most lines of rock work, since it is so essential that all the material be broken into fragments small enough to permit fast and steady operation of the shovel. In most highway work this does not mean the use of exceptionally large single charges but rather closer spacing of holes, lifts of moderate depths, and a careful proportioning and placing of the charges of the proper kind of explosive so as best to utilize the full force of the explosion in loosening and shattering the material. In very hard, tough material the holes should always be sprung so as to form a cavity or chamber of sufficient size to accommodate most of the charge at the bottom of the drill hole. The same is also true of deep holes in the softer rocks, except when the danger of losing the hole by the springing is too great. To form this cavity in hard rock may require the firing in each hole of as many as five or six gradually increasing successive charges of high-strength explosive. On most of the jobs studied one to three successive charges usually proved sufficient to spring the holes, although one single hole has a record of 16 successive charges.

Table 4 shows for a number of jobs the amount of explosives used both for springing and blasting and also certain other pertinent data as to the type of explosive, average depth of holes, and the condition of the material after the blast. The two columns showing respectively average depth and average spacing of the holes give the arithmetical averages for all holes for which complete data were available. Since sidehill cuts predominated on all but three jobs (1, 2, and 5) these figures give only a roughly approximate measure of either the actual depth or the actual spacing of the individual holes and were added only to give a general idea of the nature of the work.

Table 4 shows that the average amount of explosive used per cubic yard of rock on each of these jobs varied from 0.50 to 1.90 pounds, and that the job using the smallest amount of explosive resulted in good breakage of the material while the breakage on the job using the most explosive was very poor. This helps to emphasize the fact that good breakage or fragmentation can be obtained fairly easily in some materials, but is difficult to obtain at all in others, and especially so in shallow

cuts of tough material. The poor results on job No. 10 can be ascribed very largely to the fact that the holes were not sprung. Consequently, the charge in a 5-foot hole usually came within about 18 inches of the top, which was insufficient space for proper tamping even when 60 per cent dynamite was used. Near the close of the studies, springing the holes three times with one-half, one, and two 1½ by 8 inch sticks of 60 per cent dynamite was tried, with the net result that the use of the same total amount of explosive per cubic yard gave a better fragmentation with a spacing of 4 by 4½ feet, than had been obtained from a 3 by 3 foot spacing of unsprung holes. This wider spacing required only half the amount of drilling previously necessary.

TABLE 4.—*Average amount of explosives used per cubic yard of pay material on 17 typical jobs*

Job No.	Explosive per cubic yard				Condition of material for moving with shovel	Kind of material	Average depth of holes (approximate)	Average spacing of holes (approximate)
	Dynamite or gelatin		Black powder	Total				
	Springing holes	Blasting	Blasting					
	Pounds	*Lbs.*	*Lbs.*	*Lbs.*			*Feet*	*Feet*
1	0.04		0.85	0.89	Poor	Soft and seamy granite.	30	25 by 18.
2	.55		.98	1.53	Poor	Hard basalt	18	19 by 19.
3	.27	0.44		.71	Good	Massive granite	14	10 by 8.
4	.01		.55	.56	Fair	Conglomerate	15	Variable
5	.25	.85		1.10	Poor	Stratified limestone	16	15 by 12.
6	.06		1.06	1.12	Good	Shale	13	12 by 12.
7	.02	.03	.45	.50	Good	Disintegrated granite.	10	Variable.
8	.15	.16	.67	.98	Fair	Stratified limestone and shale.	12	10 by 10.
9	.02	.17	.50	.69	Good	Seamy basalt	21	15 by 15
10	[1]	1.90		1.90	Very poor	Hard granite	5	3 by 3.
11	.25	.75		1.00	Poor	Very hard ganite	14	8 by 8.
12	[2].55	1.00		1.55	Good	Very hard felsite	13	7 by 7.
13	.14	.59		.73	Fair	Hard schist	12	9 by 9.
14	.10	.50		.60	Good	Foliated granite	10	10 by 10.
15	.18	.75		.93	Fair	Hard granite	16	8 by 8.
16	.15	.87		1.02	Good	Very hard granite	16	8 by 8.
17	.10	.47		.57	Good	Soft granite	18	10 by 10.

[1] Near close of studies springing was begun using approximately 0.30 pound of 60 per cent dynamite per cubic yard for springing and 1.60 pounds for blasting.
[2] Part of this was used in burning the deeper holes to keep drills from sticking.

CHARGES SUGGESTED FOR FIRST TRIAL SHOTS

In shale, hardpan, and some soft or partly disintegrated rocks the use of a black powder is usually preferable, as these blasting powders are much slower in action and give a very large volume of gas which exerts a lifting or heaving action that is very effective in this type of material. The ordinary black powder can only be used in dry holes; and, since both sidehill and through cuts in road work are likely to be more or less wet, a low-strength, slow dynamite or blasting gelatin must generally be used. Figure 10 shows the results produced by an explosive charge of this type. Because of the prime importance of insuring good fragmentation, the use of a rather high-strength dynamite or blasting gelation is generally found most satisfactory for the harder rocks. For the first trial purposes, a blasting charge of 1 to 1¼ pounds of 50 per cent or 60 per cent dynamite, or its equivalent, per cubic yard of burden is suggested for untried materials of a hard and tough character. These suggestions are given in outline form in Table 5.

For the first trial only enough holes should be drilled and fired to make a fair test of what this spacing and loading will do in the given material. From this evidence either the spacing or the loading or both should be modified as the results indicate to be necessary or advisable. Table 5 is intended for trial use only where the depth of cut is between 5 and 20 feet.

TABLE 5.—*Suggestions for spacing of drill holes and amounts of explosives for the first trial shots in unfamiliar materials where hard shooting is permissible*

Type of material	Spacing of drill holes [1]		Coefficient [1] of depth of hole below grade, *c*	Pounds of explosive [2] per cubic yard of burden	Use an explosive equivalent to
	a	*b*			
Any rock exceptionally difficult to shatter.	½	½	½	1⅛	60 per cent straight dynamite.
Most hard, dense, unweathered ledge rock.	⅔	⅔	⅓	1	50 per cent straight dynamite.
All medium hard, weathered, or partly disintegrated rock; those which shatter readily and very hard shales.	¾	¾	⅓	1 to 1½	40 per cent straight dynamite, or blasting powder if holes are dry.
Ordinary shale, hardpan, and similar materials.	1¼	1	¼	⅞ to 1	Blasting powder if holes are dry; otherwise 20 per cent straight dynamite.

[1] See Figure 6 and text, p. 296.
[2] The loading suggested is net amount of explosives for actual blast and does not include amount required for springing holes.

FIGURE 10.—FOR MOST SHALES, HARD-PANS AND PARTLY DISINTEGRATED ROCKS AN EXPLOSIVE HAVING A SLOW, HEAVING ACTION IS BEST. THIS 18-FOOT LIFT WAS THOROUGHLY BROKEN, ALTHOUGH THE ONLY SURFACE INDICATIONS WERE A FEW CRACKS AND A HEAVE OR BULGE ABOVE THE ORIGINAL GROUND LINE

Following is the procedure for finding the actual amount of explosive which should be placed in each hole. Measure the depth of the hole, its distance from the next hole in the same row, and also the distance from the hole to the face or to the next row of holes. Multiply these three distances together and, if in feet, divide the product by 27. The quotient is the "burden" on this hole in cubic yards. Multiply this burden by the amount of explosive to be used per cubic yard. This gives the amount of explosive to be placed in that hole. Thus, if a hole is 13 feet deep and the spacing is 6 and 7 feet, respectively, then the burden is

$$\frac{13\times6\times7}{27}=20.2 \text{ cubic yards.}$$

If the rock is very hard and tough so that 1⅛ pounds of dynamite are required per cubic yard, then the charge for this hole will be

$$20.2\times1\tfrac{1}{8}=22.7 \text{ pounds.}$$

For shale, hardpan, or soft or partly disintegrated materials the trial charge might well be either a coarse blasting powder; if the holes are wet, a slow dynamite should be used. If the ground will permit springing without the loss of too many holes they should always be sprung sufficiently to chamber practically the entire charge and should be loaded and tamped as indicated in Figure 11.

When there are intervening or alternating strata of hard and soft material, closer than normal spacing will usually be required and a portion of the charge in the form of dynamite should be placed in the stem of the hole within the hard strata as indicated in Figure 11. These auxiliary charges in the stem should all be fired simultaneously with the main charge by means of independent caps and primers wired to the same circuit. This method should give fairly satisfactory results as far as fragmentation is concerned, provided that the greatest spacing of the holes and rows in fairly hard or tough material is not more than three-quarters the depth of the cut, and that these holes penetrate sufficiently below grade to leave no hard, projecting points between adjacent holes. If the trial blast shows insufficient fragmentation the amount of explosive may be increased, or if much of the material is broken too large the spacing should be decreased. Since the burden on any hole of a given depth varies directly as the product of the spacings between holes and rows, the amount of explosive placed in each hole should be varied proportionally whenever the distance between holes and rows is increased or decreased.

Special care should always be taken to make certain that the holes along the ditch or slope lines are placed and spaced correctly and are of sufficient depth to insure full breakage to the bottom of the established ditch line. In deep cuts of difficult materials where the requirements as to slope smoothness are severe it is frequently advantageous to drill the slope or ditch holes at about the same angle as the specified side slope. Then, instead of placing the entire charge at the bottom of the hole a part is placed in the stem as a secondary charge, which is primed and wired so as to explode simultaneously with the main blast.

CLOSER SPACING NECESSARY WHERE HEAVY SHOOTING IS NOT PERMISSIBLE

Where heavy shooting is not permissible the only way in which good fragmentation can be obtained is by a closer spacing of the holes. But drilling is expensive. A foot of drill hole seldom costs less than a pound of high-grade explosive and frequently as much as 2 pounds. The spacing should therefore be the greatest at which sufficient fragmentation to permit easy shovel operation can be achieved within the limits of the heaviest permissible shooting. In very hard, tough rock this maximum permissible spacing is sometimes as low as one-half and seldom exceeds three-quarters the depth of the cut for cuts less than 16 feet deep, and probably rarely exceeds 12 feet of actual distance for cuts over 16 feet, unless supplementary charges are placed in the stem of the original or master holes and also in holes which have been drilled to within one-third to one-half the depth of the cut near the middle of each quadrilateral formed by four deep holes.

SUMMARY OF FACTS BROUGHT OUT BY STUDY

The sole purpose of drilling and blasting in highway grading work is to condition the material for removal, usually with a power shovel. Moreover, the daily or hourly cost of the power-shovel grading outfit is high—usually between $15 and $25 an hour—and practically constant whether the rate of production is high or low. Low production cost can therefore only be obtained through high production; but in order to maintain a high production rate the blasting must be thorough and

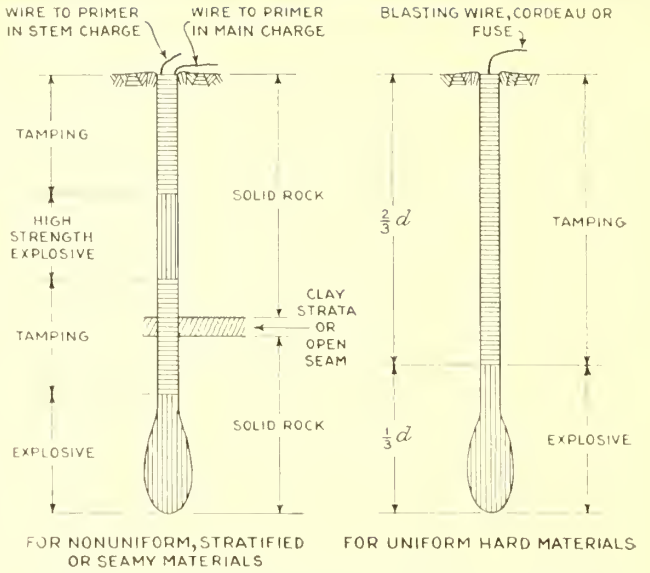


FIGURE 11.—SUGGESTIONS FOR LOCATING THE BLASTING CHARGE. HOLES SHOULD BE SPRUNG UNTIL THE DESIRED AMOUNT OF EXPLOSIVE WILL NOT OCCUPY MORE THAN ONE-THIRD OF TOTAL DEPTH. IN MATERIAL WHICH IS HARD TO SHATTER PROPERLY, A SMALL CHARGE OF HIGH-STRENGTH EXPLOSIVE SHOULD ALSO BE PLACED IN THE STEM WHENEVER THE DISTANCE TO THE NEAREST HOLE EXCEEDS 8 FEET. NO STEM CHARGE SHOULD HAVE LESS THAN 2 FEET OF TAMPING OVER IT

sufficient to shatter the material into fragments small enough for ready handling by the shovel. The blasting must also proceed sufficiently ahead of the shovel so that there will be no appreciable delay from waiting for the blasting to be done. This means, in general, that the blasting of each cut should be completed before work is begun on that cut by the shovel. On this account, road work seldom affords much opportunity for firing the various rounds against a free face as in quarry work. As a result, relatively more explosive is required and relatively closer spacing of holes is necessary in order to insure proper fragmentation.

Perhaps the most significant feature disclosed by this study of drilling and blasting in highway grading work is the very general absence of that grade of supervision, organization, and matured planning which is given to the remainder of the job. Frequently all decision as to such important points as the location, spacing, and depth of holes is left entirely to the drillers. Too often this results in the holes being placed where it is convenient to drill rather than where they will do the most good. This lack of planned direction usually also extends to the method of loading and the amount of explosives used in each individual hole. With such conditions forming the general rather than the exceptional practice, there is no wonder that nearly three-quarters of all rock jobs reported the shooting as unsatisfactory from the viewpoint of fast, easy shovel operation. All the available data indicate that the cost of this poor shooting is as high and (if we include the usual pop holes to attain grade and the block holes and mudcaps necessary to break up boulders) frequently much higher than on those jobs where the shooting is good. For similar conditions and materials, the poorly-shot jobs often actually pay more per cubic yard for their drilling and blasting than do those jobs on which

(Continued on p. 308)

EFFECT OF TYPE OF BREAKING MACHINE ON THE MODULUS OF RUPTURE OF 6 BY 6 INCH CONCRETE BEAMS

By O. K. NORMANN, Junior Highway Engineer, Division of Management, U. S. Bureau of Public Roads

CONCRETE pavements are now very generally opened when the breaking strength or modulus of rupture of beams made at the time the slab was placed reaches some specified value. Various types of portable or semiportable machine are used for breaking these beams, and the question of the comparative values given by different types of machine under actual field conditions is therefore of prime importance.

During the course of some rather extensive production studies during the summer of 1930, an opportunity arose to compare the values obtained under field conditions from two radically different types of beam-breaking machine, and to check these results against the values given by 2-point loading in a universal testing machine. These data indicate that it would be well if every field machine could be calibrated against the standard laboratory set-up before it is placed in service. As these field machines frequently receive very rough treatment it might also be desirable to have this calibration rechecked before the beginning of each subsequent construction season.

The chief structural difference in the various types of machines now in use for breaking concrete beam test specimens is in the arrangement of the lever arms, which produce the bending moment in the beam. All of these various arrangements are apparently satisfactory so far as breaking the test specimens is concerned, but differ widely in the amount of shearing stress imparted while developing the necessary bending moment.

TESTS MADE WITH THREE TYPES OF BREAKING MACHINE

During the progress of certain production studies on Tennessee Federal-aid project 212–A, in McNairy County, involving the construction of 12.34 miles of concrete pavement, it was noted that the modulus of rupture as obtained for beams broken by a portable multiple-lever beam-breaking machine was regularly much higher than values given by beams broken with a very simple cantilever device. Since two multiple-lever machines, one simple cantilever device, and a universal testing machine were available, a series of eight sets of beams were at once made and cured under conditions as nearly alike as possible in every respect. Some breaks of each set were made by the simple cantilever device; others by one or the other of the two portable multiple-lever machines, and still others were broken by the 2-point loading method in the universal testing machine.

Three 6 by 6 by 36 inch beams for each of 7 sets and one set of 4 beams, or a total of 25 beams, were made from concrete taken from near the center of selected batches of uniform consistency as they were dumped on the subgrade. The molds, made of angle irons with an iron-plate bottom, were each filled, spaded once around the sides, tamped 50 times 1 inch from the side with a standard tamping rod, and then spaded once more, after which the top surface was finished with a trowel. After being marked and numbered, the beams were covered with wet burlap and allowed to remain undisturbed in the molds until they had been hauled to the curing tank on the following day, when the molds were removed and the beams placed under water.

Beam 1 of each set was kept submerged until the twenty-eighth day, when it was removed from the curing tank and broken (three breaks) on the simple cantilever device. Beams numbered 2 and 3 of each set were removed from the curing tank after 23 to 26 days of curing, wrapped in wet burlap, and hauled by truck to the Tennessee State highway laboratories, where they were placed in the moist room until the twenty-eighth day, at which time each beam was broken once in the 50,000-pound universal testing machine and then twice in the multiple-lever type of portable beam-breaking machine. The top, or troweled surface, of the beam was always placed so as to be on the tension side.

CANTILEVER APPARATUS DESCRIBED

The cantilever apparatus used in these tests, as well as in breaking a large number of other similar beams, consisted essentially of (1) a strong upright post provided with a heavy bearing plate and a simple clamp for holding the beam firmly in a horizontal position while the bending moment was being applied; and (2) a 6-foot lever arm, one end provided with an adjustable socket for attachment to the beam and the other with a hook for suspending a 50-gallon tank to which water was added at a uniform rate until the breaking point of the beam was reached. The exact arrangement of the apparatus as used on this work is shown in Figure 1. Figure 2 shows the details of a similar apparatus which includes such modifications as three months of experience in making over 500 breaks of 8 by 6 inch beams with the first machine suggested as desirable.

The improvised machine used on this job to break the beams in the field was of no standard design or make. The advantages of this type of machine are:

(1) Simplicity of design, portability, and cheapness.

(2) The fact that the load may be applied at a uniform rate, or at any desired rate.

(3) The fact that the beam arm hangs absolutely free and is not dependent upon any ball or roller bearings. There is no system of levers that may vary in length or position. The only variable is the total weight of the water. This may be weighed to the nearest pound.

The method of operation was very simple. The concrete beam was clamped to a bearing plate on top of the wooden post by means of two ½ by 3 inch steel plates. The large bearing plate could be adjusted by means of the bolts holding it, so as to be level. The rear clamp was not tightened very hard. If it had been, there would have been a tendency for the beam not to be in contact with the bearing plate directly under the plane of maximum stress. For this reason, both of the clamps were left slightly loose in order that the beam would always be in direct contact with the breaking edge. The use of a heavy but rather narrow angle iron lapped into the front top edge of the post is suggested for future machines of this kind to replace the large bearing plate, while the two top clamp plates should be replaced by one similar angle iron. This modification is shown in Figure 2.

The beam arm was held in place by two ½ by 3 inch plates, one on the top and one on the bottom of

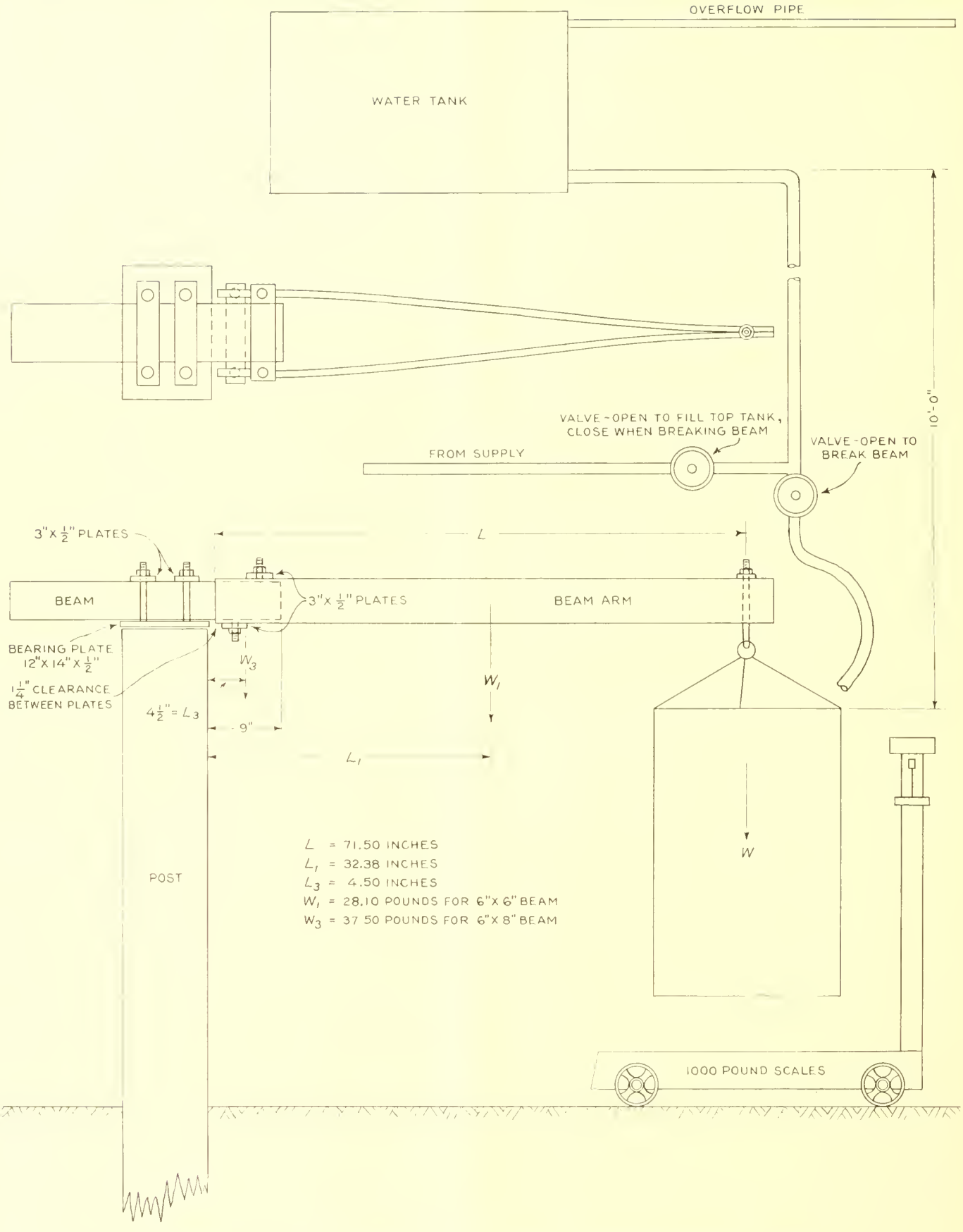


FIGURE 1.—DIAGRAM OF IMPROVISED SINGLE-LEVER FIELD MACHINE AS USED IN TESTS

the arm. The plates could be adjusted so that the beam arm would be level at the time of the break. At first the total deflection of the end of the beam arm was about 5 inches when the break took place. This defleetion was later reduced to about 3 inehes by placing angle irons at the end of the wooden arm, thereby redueing the play in the two plates.

Water was supplied at a uniform rate from a tank above the beam machine. This tank was filled to the same level before making each break. There was a very slight difference in the rate of application of the load at the beginning and end because of the fact that the head was reduced slightly as water was drawn from the top tank. An inflow equal to the outflow would insure a more constant rate of load applieation and would also permit the use of a smaller tank.

A wooden platform was plaeed between the scales and the tank so that the heavy tank of water would not fall directly on the scales. The tank of water could then be easily lowered on to the scales and weighed to the nearest pound. If scales are not readily available, the tank can be calibrated and the quantity measured within an accuracy of about 1 per cent. In this case a mound of sand or earth should be made so that the drop of the tank will not exceed 1 inch when the beam snaps.

Following is the method used in eomputing the modulus of rupture for tests with the single-lever breaking device:

In Figure 1

W = Load applied by tank.
L = Corresponding moment arm.
W_1 = Weight of beam arm and auxiliary parts.
L_1 = Corresponding moment arm.
W_3 = Weight of cantilevered portion of test beam.
L_3 = Corresponding moment arm.

Then, if M is the moment acting at the support,

$$M = WL + W_1L_1 + W_3L_3,$$

and the modulus of rupture is given by the equation,

$$S = \frac{Mc}{I} = \frac{6M}{bd^2}.$$

LOAD APPLIED AT THIRD POINTS ON UNIVERSAL TESTING MACHINE

A frame had been made by the Tennessee highway department to support a 6 by 6 inch beam, as shown in the diagram in Figure 3, so that the 50,000-pound universal testing machine could be used to break beams. The beam was supported at each end on 2-ineh iron plates, and the load applied at two points on the beam by the testing machine. Cylindrical rollers were placed between the plates in contact with the beam, as well as between the plates connected with the frame through which the load was applied.

The apparatus for breaking the beams was built so that the distances between the bearing points could be changed if desired. For this study the outside supports were placed 30 inehes apart, with the load applied at the two one-third points. The bending moment due to applied load was then the same for all points between the two places where the load was applied, thus providing a favorable eondition for the break to occur at the weakest place within the middle 10 inches of the beam. Furthermore, while the bending moment was maximum and uniform within this section of the beam, the shearing stress due to applied load was equal to zero over the full length of the same distance. The troweled surface was placed on the bottom so that it would be on the tension side of the beam when it was broken.

Values of the modulus of rupture for tests made in the universal machine were computed by the following formula:

$$S = \frac{3WA}{bd^2} + \frac{3W_1L}{4bd^2},$$

where W is the total applied load, A the distance from each support to the adjacent loading point, W_1 the weight of the beam, and L the distance between supports. Since in these tests the loading was at the third points, the formula was reduced to the form

$$S = \frac{L\ (W + \frac{3}{4}W_1)}{bd^2}.$$

MULTIPLE-LEVER TESTING MACHINES DESCRIBED

Two of these machines were available, one in the laboratory and one in the field. All of the beams made for this particular study to be broken on these machines were broken in the laboratory and with the laboratory machine, except two, which were broken on the field machine. It was not intended to break any on the field machine but the results obtained on the other machines made it appear worth while to carry the study further, because there was considerable difference in the amount of shear produced in the two machines.

In both of the multiple-lever machines the load was applied by a cable attached to a hand wheel. A dynamometer between the cable and the end of the beam arm indicated the load being applied. This dynamometer was carefully tested from time to time, and was always correct within less than 1 per cent.

The chief difference between the laboratory and the field machine is the distance from the point where the load was applied to the plane of maximum bending stress. In the laboratory machine this distance is 5.5 inches, while in the field machine it is 8.75 inches. This difference causes a greater shearing stress in the one case for the same moment at the plane of maximum stress. The essential differences of the two machines are shown in Figure 4.

Following is the method used in computing the modulus of rupture for tests with the multiple-lever machines.

In Figure 4,

W_1 = Applied load.
L_1 = Distance from the upper fulcrum to the point of load application.
W_2 = Weight of beam arm.
L_2 = Corresponding moment arm with respect to upper fulcrum.
W_3 = Weight of cantilevered portion of test beam.
L_3 = Distance between upper and lower fulcrums.
W_4 = Weight of dynamometer.
L_4 = Moment arm of lower fulcrum with respect to front edge of supporting post.
L_5 = Length of cantilevered section of test beam.

The reaction at the lower fulcrum is given by the equation,

$$R = \frac{W_1L_1}{L_3} + \frac{W_2L_2}{L_3} + \frac{W_4L_1}{L_3}$$

The moment at the support is then given by the equation,

$$M = \frac{W_1L_1L_4}{L_3} + \frac{W_2L_2L_4}{L_3} + \frac{W_4L_1L_4}{L_3} + \frac{W_3L_5}{2}$$

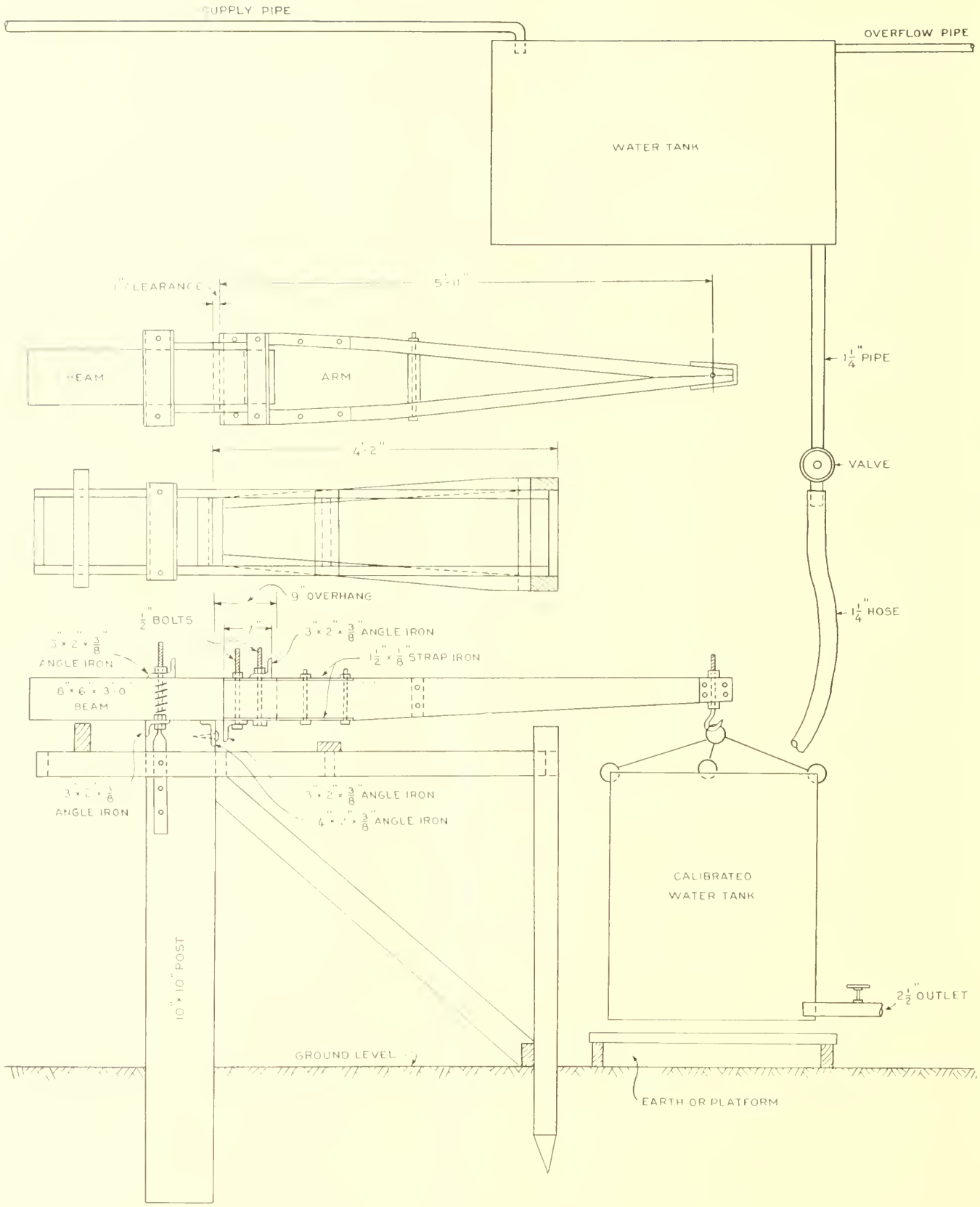


FIGURE 2.—DIAGRAM OF IMPROVED SINGLE-LEVER FIELD MACHINE

and the modulus of rupture is obtained from the relation

$$S=\frac{6M}{bd^2}$$

VALUES OF MODULUS SHOW SIGNIFICANT DIFFERENCES

Values of the modulus of rupture of all beams tested in the three types of machine are given in Table 1. Table 2 gives the differences between average values for each set tested on the universal machine and the corresponding average values for tests with the single-lever and multiple-lever machines.

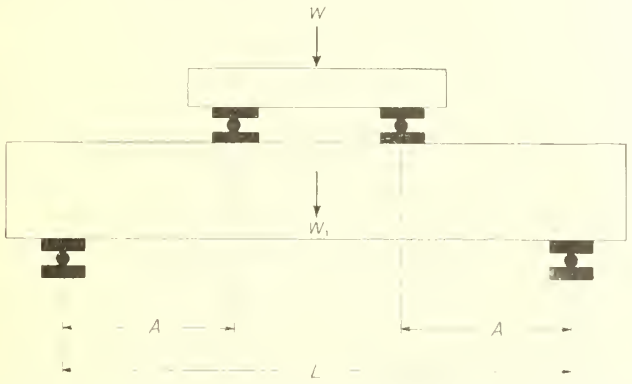


FIGURE 3.—DIAGRAM SHOWING 2-POINT LOADING OF BEAM IN UNIVERSAL TESTING MACHINE

TABLE 1.—*Comparison of values of modulus of rupture given by the three types of beam-breaking machine; modulus of rupture given in pounds per square inch*

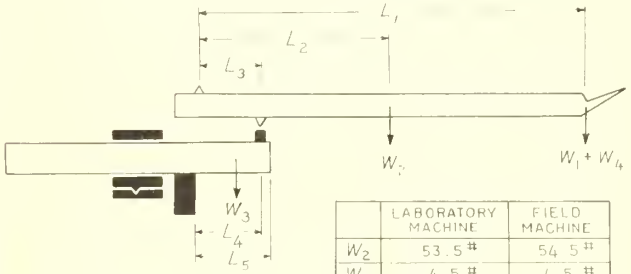
Set No.	Improvised single-lever machine				Universal machine			Multiple-lever laboratory machine					
	Beam 1, break No.—			Average	Beam 2 Break No.—	Beam 3 Break No.—	Average	Beam 2, break No.—			Beam 3, break No.—		Average
	1	2	3		1	1		1	2	3	1	2	
5	716	614	730	687	712	666	689	805	823	860	926	841	863
6	553	563	568	561	525	559	542	759	764	....	883	780	797
9	572	575	540	562	570	611	590	683	672	....	761	868	746
12	574	586	536	565	614	576	595	756	772	773	786	781	773
14	560	517	519	532	605	546	575	856	816	....	710	786	792
16	528	654	613	598	572	565	568	788	732	....	812	867	800
20	628	633	590	617	644	569	607	950	913	....	938	899	925
Average for machine				589			595						814

TABLE 2.—*Differences in average values of modulus of rupture given by tests with universal testing machine and corresponding values given by tests with the improvised single-lever machine and the multiple-lever laboratory machine*

Set No.	Improvised single-lever machine	Multiple-lever laboratory machine
5	−2	+174
6	+19	+255
9	−28	+156
12	−30	+178
14	−43	+217
16	+30	+232
20	+10	+318
Average	23	219

The results obtained from each machine were very consistent for both the individual and the average breaking strengths. The real differences lie between the values given by the different types of breaking machine. The improvised field machine gave results very close to those given by the universal machine, the difference between the averages for all the breaks being only 6 pounds per square inch. For these two machines the maximum difference between the average strengths of any set of beams, as shown in Table 2, was only 43 pounds, with an average difference of only 23 pounds. The maximum and minimum strengths on the improvised field machine were also very close to the maximum and minimum strengths on the universal.

The laboratory multiple-beam machine gave much higher results than either the universal or the improvised field machine. The increase in average strength was 219 pounds, or 36.8 per cent over the average strength given by the universal testing machine. On the individual sets of beams the average strength was from 156 to 318 pounds per square inch higher. There was not a break made on the multiple-beam machine that was as low as the highest breaks obtained for corresponding sets on either of the other machines. The smallest and the largest difference between the lowest break on the multiple-beam machine and the highest break on the other two, for beams of the same set, were 61 pounds and 255 pounds, respectively.



	LABORATORY MACHINE	FIELD MACHINE
W_2	53.5 #	54.5 #
W_4	4.5 #	4.5 #
L_1	48.5 "	48.0 "
L_2	18.3 "	16.8 "
L_3	3.25 "	7.00 "
L_4	5.50 "	8.75 "
L_5	6.75 "	10.00 "

FIGURE 4.—DIAGRAM SHOWING LOADS AND MOMENT ARMS ACTING IN MULTIPLE-LEVER MACHINES

The results of the study showed that the multiple-beam machine gave results so much larger than those obtained on the other two machines that it was decided to carry the study to the multiple-beam field machine. The essential difference between the multiple-beam laboratory machine and the multiple-beam field machine is the distance from the plane of maximum stress to the point at which the load is applied to the test beam. In the first case this distance is 5.5 inches, and in the second it is 8.75 inches.

Results of tests of two beams each from set 19 on the improvised field machine and the multiple-beam field machine are given in Table 3.

All of these beams were made from the same batch. Beams 1 and 4 were tamped 50 times, and beams 2 and 3 were tamped 180 times.

The excess in average strength given by the multiple-beam field machine was 152 pounds, or 26 per cent higher than the average strengths obtained on the improvised field machine.

TABLE 3.—*Comparison of values of modulus of rupture given by tests with the multiple-lever field machine and the improvised single-lever machine, for two beams each from set 19*

Beam	Multiple-lever field machine			Improvised single lever-machine			
	Break number		Average	Break number			Average
	1	2		1	2	3	
1				550	552		551
2				639	577	617	611
3	753	770	762				
4	693	715	704				
Average for machine		733			581		

CONCLUSIONS SUMMARIZED

In view of these facts the following observations and conclusions seem justified:

1. The breaks on the beams broken in the multiple-lever machines showed that a much larger percentage of the rock was broken than in the other machines.

2. The type of machine used to make the break will determine to a large extent what breaking strength will be obtained.

3. The results show that the apparent quality of concrete, as indicated by test specimens, may be varied be merely changing the apparatus for breaking specimens. They may also help to emphasize why a more standard method of making and especially of testing beams in the field should be adopted.

4. It would be advantageous if every field machine could be calibrated against the standard laboratory set-up before it is placed in service. As these field machines frequently receive very rough treatment it might also be desirable to have this calibration rechecked before the beginning of each subsequent construction season.

Acknowledgement is made to the Department of Highways and Public Works of the State of Tennessee, without whose valuable assistance and active cooperation in both laboratory and field these data could not have been obtained.

(Continued from p. 302)

the shooting is good, and in addition carry the penalty of a low rate of production on the part of the shovel. The conclusion therefore seems inescapable that, under present close margins of bidding, the grading contractor who wishes to remain in business can not afford to permit poor shooting on his rock jobs.

ROAD PUBLICATIONS of the BUREAU OF PUBLIC ROADS

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Report of the Chief of the Bureau of Public Roads, 1927.
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Report of a Survey of Transportation on the State Highways of New Hampshire. (1927.)

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Report of a Survey of Transportation on the State Highways of Pennsylvania. (1928.)

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

JANUARY 31, 1932

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal-aid allotted	MILEAGE Initial	MILEAGE Stage	MILEAGE Total	Estimated total cost	Federal-aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total		
Alabama	2,405.4	$ 1,541,469.27	$ 748,838.58	62.6		62.6	$	$				$ 5,389,119.31	Alabama
Arizona	1,113.6	3,100,414.28	2,038,617.71	151.7	84.6	236.3	214,260.26	161,316.64	18.7		18.7	1,491,162.58	Arizona
Arkansas	1,919.7	3,795,361.89	1,772,437.69	86.0	72.7	158.7	362,032.53	181,016.26	17.3		17.3	1,991,762.04	Arkansas
California	2,214.2	10,488,440.29	4,635,262.61	222.0	44.8	266.8	2,245,227.41	1,086,831.69	47.1	14.1	61.2	1,804,409.96	California
Colorado	1,516.8	5,212,372.26	2,662,850.29	221.9	66.0	287.9	647,326.44	269,751.14	16.3		16.3	2,067,570.68	Colorado
Connecticut	288.5	3,084,244.77	1,151,767.90	22.9		22.9						713,262.14	Connecticut
Delaware	361.6	154,414.00	77,207.00	4.3		4.3	465,212.50	232,606.25	25.6		25.6	338,081.68	Delaware
Florida	681.7	5,239,072.77	2,432,196.81	147.0		147.0	49,004.56	24,502.28	.1		.1	3,021,969.03	Florida
Georgia	2,989.0	6,988,646.29	3,250,027.65	184.6	153.5	338.1	988,337.56	494,168.76	38.0	17.2	55.2	1,744,179.93	Georgia
Idaho	1,486.3	1,726,513.36	998,918.80	86.9	80.3	166.2	252,882.85	152,007.29	18.0		18.0	1,423,480.58	Idaho
Illinois	2,627.1	22,713,061.29	10,563,419.39	665.8	28.7	694.5	3,736,850.76	1,642,351.35	140.2	9.0	149.2	2,905,293.87	Illinois
Indiana	1,787.9	7,156,748.04	3,539,673.88	189.2		189.2	3,897,200.83	1,789,777.42	141.4	2.9	144.3	1,584,606.30	Indiana
Iowa	3,379.0	331,333.50	138,397.76		11.9	11.9	1,743,306.44	826,688.01	61.3	18.0	79.3	1,973,117.19	Iowa
Kansas	3,649.7	3,006,616.48	1,391,823.86	161.2	16.3	176.5	2,923,789.57	1,427,002.36	188.8	6.4	194.2	1,760,691.19	Kansas
Kentucky	1,871.7	2,348,616.36	1,067,373.06	137.2	4.0	141.2	577,471.72	269,284.62	13.3	41.6	54.9	1,727,826.78	Kentucky
Louisiana	1,541.7	7,216,064.49	2,803,668.96	1.2.6	10.6	123.2	1,295,607.93	69,270.07	.5		.5	1,666,879.62	Louisiana
Maine	706.0	1,804,346.10	789,080.76	31.4		31.4	37,546.34	14,085.00	.9		.9	1,318,971.43	Maine
Maryland	770.1	441,461.15	149,608.89	13.4		13.4	250,874.75	125,437.37	11.8		11.8	785,651.09	Maryland
Massachusetts	803.2	7,747,403.42	2,949,352.76	70.1		70.1	784,690.49	348,347.37	8.6	.2	8.8	687,103.02	Massachusetts
Michigan	1,974.3	10,437,694.69	4,623,057.69	374.3	58.2	432.6	800,781.24	378,823.37	34.0	8.6	42.6	3,168,443.32	Michigan
Minnesota	4,291.3	2,464,947.82	876,785.45	41.1	58.7	99.8	7,747,946.74	2,474,985.76	157.5	201.9	359.4	150,028.10	Minnesota
Mississippi	1,792.4	4,014,998.22	1,968,703.64	169.4	78.0	247.4	103,524.13	61,762.05	9.5		9.5	5,383,500.51	Mississippi
Missouri	2,851.2	2,333,230.66	975,887.91	62.8	3.4	66.2	1,560,466.24	703,143.85	60.1	3.5	63.8	3,047,368.46	Missouri
Montana	2,813.9	5,417,980.89	3,034,529.24	442.7	22.8	466.5	403,394.86	224,368.61	61.4	.5	61.9	3,253,079.46	Montana
Nebraska	4,152.9	4,920,915.22	2,406,758.29	206.5	76.6	283.1	84,279.68	41,975.64	2.1		2.1	2,647,063.39	Nebraska
Nevada	1,345.8	974,327.23	744,741.76	30.4	89.8	120.2						1,658,863.00	Nevada
New Hampshire	417.6	676,512.12	265,632.44	13.4	2.6	15.9						547,810.24	New Hampshire
New Jersey	682.1	5,708,263.62	1,976,622.53	69.5	.6	60.1						1,604,603.13	New Jersey
New Mexico	2,177.7	2,748,137.66	1,788,348.90	136.7	13.8	149.5	481,698.71	303,425.61	14.2	10.4	24.6	1,260,679.38	New Mexico
New York	3,239.0	12,643,900.00	5,610,500.00	282.2		282.2	3,614,800.00	1,350,000.00	68.7		68.7	4,436,572.98	New York
North Carolina	2,221.8	1,348,388.13	653,221.40	59.9		59.9	245,906.23	122,953.10	17.4	4.0	21.4	3,858,352.40	North Carolina
North Dakota	6,118.2	1,828,691.83	918,958.67	160.4	171.8	332.3	1,327,226.48	679,129.08	149.7	259.5	409.2	2,037,725.62	North Dakota
Ohio	2,829.5	7,002,996.22	2,423,620.10	100.1	29.8	129.9	1,173,200.00	572,962.46	10.7	.4	11.1	4,086,869.51	Ohio
Oklahoma	2,184.1	5,179,760.02	2,522,739.07	137.4	83.4	220.8	498,297.51	217,779.48	10.8		10.8	2,328,366.69	Oklahoma
Oregon	1,545.6	3,287,004.12	1,835,226.98	84.0	35.7	119.7	454,179.00	237,632.02	12.8	11.1	23.9	1,896,364.68	Oregon
Pennsylvania	3,012.1	4,193,848.20	1,917,622.88	45.8		46.8						5,096,998.06	Pennsylvania
Rhode Island	257.5	811,653.61	427,281.06	16.6		16.6	81,305.62	40,652.81	2.2		2.2	329,207.16	Rhode Island
South Carolina	1,997.0	2,579,138.08	1,148,707.78	63.0	43.6	106.6						1,375,312.87	South Carolina
South Dakota	4,035.8	2,935,366.20	1,625,070.82	174.8	175.3	350.1	497,613.45	300,502.56	54.3		54.3	1,267,439.45	South Dakota
Tennessee	1,662.4	848,984.19	422,367.41	24.2	8.7	32.9	437,058.09	202,974.46	13.6	6.4	20.0	3,883,841.72	Tennessee
Texas	7,590.0	13,052,766.47	5,861,194.84	653.6	163.0	816.6	2,728,062.52	1,234,664.59	197.3	46.8	244.1	6,787,271.18	Texas
Utah	1,218.6	650,176.96	452,772.23	59.5	.4	59.9	367,710.43	272,080.68	23.1	3.3	26.4	1,294,723.37	Utah
Vermont	339.1	104,832.58	62,345.06	4.3		4.3						572,077.12	Vermont
Virginia	1,861.3	2,748,074.66	1,320,165.66	144.2	16.3	160.5	160,977.42	80,488.71	5.4		5.4	2,090,486.72	Virginia
Washington	1,195.4	1,927,757.38	947,168.66	56.9		56.9	566,682.40	222,000.00	27.5		27.5	1,779,628.99	Washington
West Virginia	884.0	1,735,051.39	761,625.34	44.6	10.5	55.1	187,685.92	86,032.31	.7		.7	1,220,976.79	West Virginia
Wisconsin	2,664.8	3,119,586.51	1,166,563.12	138.6	14.3	152.8	90,647.32	45,000.00		3.5	3.6	2,681,895.81	Wisconsin
Wyoming	2,022.1	2,522,816.68	1,492,230.46	173.6	88.9	262.5	384,014.48	246,606.78		47.4	47.4	785,801.14	Wyoming
Hawaii	63.1	974,113.04	445,557.39	30.0		30.0	82,324.99	22,869.00	1.5		1.5	2,019,071.60	Hawaii
TOTALS	100,262.6	203,386,392.15	93,715,661.81	6,669.2	1,818.6	8,377.8	44,401,309.30	19,226,256.49	1,682.4	715.6	2,398.0	106,844,531.07	TOTALS

[1]The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement

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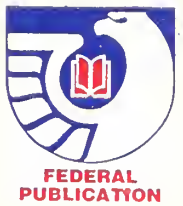

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